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INSTITUTED 1852.

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594.
(Vol. XXVIII.—May, 1893.)

RAINFALL AND RIVER-FLOW.

By CYRUS C. BABB, Jun. Am. Soc. C. E.

The futility of basing computations of discharge for any river upon percentages of the average monthly rainfall in its basin has become pretty well recognized. It is desirable, therefore, that another method, giving similar results and with a fair degree of accuracy, be found if possible. The results of any stream gaugings, to be of value, necessarily require that they cover a considerable period of time, which is generally beyond the limits of the average engineer to carry out, both financially and in point of time. A great misconception exists in the minds of many engineers as to the percentage of the annual rainfall that finds its way into the rivers. One western engineer that the author knows of used in designing certain works for irrigation as high as 60%, when one-half of it would have been nearer

the truth. Of course, such errors are the due result of lack of information and of data upon the subject in question.

The present paper seeks to bring out certain facts in regard to the flow of streams as shown by the gaugings of rivers in the United States. All of the available data bearing upon the subject, much of which has never been published before, were compiled, and, together with the results of certain original investigations by the author, were digested for this article. A method is given for estimating monthly discharges based upon the percentages of annual run-off to rainfall, and upon the distribution or percentages by months of the entire flow for the year for different basins in the United States. A brief description of the controlling factors of discharge for several drainage basins will be given.

The Connecticut River rises in northern New Hampshire and flows in a generally southern course for about 375 miles until it reaches the sea. The total drainage area is 11 083 sq. miles, apportioned among the different political divisions as follows: Canada, 156 sq. miles; New Hampshire, 2 986; Vermont, 3 810; Massachusetts, 2 705, and Connecticut, 1 426 sq. miles. The river flows through a glaciated region, and as a natural consequence the basin contains a large number of lakes and springs that tend to regulate the discharge, reducing the intensity of floods and increasing the summer flow.

The topography of the basin is hilly and even mountainous. The older rocks, granites, gneisses and schists, are found in the mountains; while the glacial drift, composed of clays and sands interspersed with boulders of all sizes, occurs in the valleys. One notable formation is the belt of triassic red sandstone extending from the northern Massachusetts boundary south to the coast.

Numerous rapids and falls occur throughout the course of the river. From its source to the New Hampshire-Massachusetts State line, a distance of 240 miles, there is a total fall of 1 830 ft., giving a grade of 7.6 ft. per mile. From there to the mouth, a distance of 135 miles, the total fall is 205 ft., or 1.5 ft. per mile. For its entire length the fall averages 5.4 ft. per mile. In Vermont and New Hampshire the river drains elevations of from 4 000 to 6 000 ft.

The investigations of the Engineer Corps, U. S. A., in this basin, give results of great value for the study of this and similar water-

sheds. Their discharge measurements cover a period from 1871 to 1879, with daily gauge-height records to end of 1885 and partial ones since that date to the present time. Access was had to consider ble unpublished daily rainfall data of the Weather Bureau which are also incorporated into this article. Table No. 1 gives by months and years the average rainfall, the average discharge, and the relation between these two, or the percentage of discharge to rainfall, for the Connecticut basin from 1871 to 1885, inclusive, except for 1882 and 1883, for which the data could not be obtained. Column 1 gives the means of the records of 12 rainfall stations distributed quite uniformly throughout the basin. Column 2 the monthly run-off in inches, *i. e.*, if the total flow for the month were spread over a plain of the size of the drainage area at the point of measurement it would cover it to the depth specified in the table. Column 3 gives the percentage of discharge to monthly rainfall.

TABLE No. 1.

MONTH.	1871.			1872.			1873.			1874.		
	Rain. Inches.	Flow. Inches.	Percent.	Rain. Inches.	Flow. Inches.	Percent.	Rain. Inches.	Flow. Inches.	Percent.	Rain. Inches.	Flow. Inches	Percent.
January...	......	......	......	1.70	1.23	72.5	3.97	2.71	68.2	4.21	5.70	135.6
February..	2.60	1.87	72.0	2.10	.77	36.7	2.82	1.90	67.4	2.68	4 06	151.6
March. ...	4.30	5.64	131.1	2 50	.90	36.0	3.81	1.82	47.8	2.17	3.00	138.4
April	3.20	2.74	85.6	1.61	5.50	344.0	1.85	7.61	412.3	6.52	2.61	40.0
May........	3.40	3.42	100.0	4 41	3.46	78.5	2.70	5.84	217.0	3.95	5.86	146.9
June......	3.50	.99	28.3	5.56	2.64	47.5	1.68	1.18	70.3	4.76	3.11	65.4
July	4.10	.69	16.8	5 49	1.13	20.6	5.28	.80	15.2	6.09	2 32	38.2
August....	5.80	.80	13.8	7.91	2.52	32 1	3.15	.73	23.2	3.52	1.19	33.8
Septemb'r.	1.50	.85	56.7	4.33	2.24	51.8	4.63	.74	16.0	2.74	.73	26.7
October...	3.70	1.02	27.6	3.62	1.76	48.6	6.84	2.81	41.1	2.81	.81	28.9
November.	3.00	1.65	55.1	4.47	2.64	59.1	3.57	1.67	46.8	2.21	.67	30.4
December.	2.60	1.44	55.4	3.01	1.92	63.8	3.55	2.81	79.2	1.58	.75	47.6
	37.70	21.11	56.2	46.71	26.71	56.6	43.85	30.62	69.9	43 24	30.81	71.4

MONTH.	1875.			1876.			1877.			1878.		
	Rain. Inches.	Flow. Inches.	Percent.	Rain. Inches.	Flow. Inches.	Percent.	Rain. Inches.	Flow. Inches.	Percent.	Rain. Inches.	Flow. Inches.	Percent.
January...	3.24	.72	22.2	2.48	2.98	120.1	2 61	.73	31.6	3.87	1.66	42.9
February .	3.31	1.13	34.2	5 02	2.70	53.8	.79	.78	99.0	3.42	2 04	59.6
March.....	3.77	2.05	54.3	6.77	3.94	58.2	7.02	3.91	55.7	3.10	3.65	117.6
April......	2.76	6.14	222.1	3.21	6.74	210.0	2.77	4.64	167.5	6.78	5.15	76.0
May.......	2.85	4.68	164.5	3.79	6 64	172.8	1.07	1.90	177 6	3.14	3 57	113.8
June......	4.44	1.49	33.6	4 44	1.72	38.8	4.64	.88	19.0	5.07	1.51	29.8
July	3 66	.96	26.2	6.10	.93	15.3	4.76	1.07	22.5	3.72	.85	22.9
August....	6.45	1.34	20.8	1.97	.70	35.6	4.60	.96	20.9	4.80	1.09	22.8
Septemb'r.	3.41	.73	21 4	5 30	.72	13.6	1.45	.76	52.5	2.76	.84	30.4
October ...	4.44	1 15	25.9	2.14	.71	33.2	5.79	1.33	23.0	3.38	.77	22.8
November.	3.51	1.72	49.0	3.13	.85	27.2	5.84	3.18	54.5	4.42	1.45	33.0
December.	1.23	1.84	149.5	3.83	.72	18.8	1.58	1.95	123.4	5.74	4.93	86.0
	43.07	23.95	55.6	48.18	29.15	60.6	42.92	22.09	51.5	50.20	27.51	54.7

MONTH.	1879.			1880.			1881.		
	Rain. Inches.	Flow. Inches	Per-cent.	Rain. Inches.	Flow. Inches.	Per-cent.	Rain. Inches.	Flow. Inches.	Per-cent.
January......	3.03	1.58	52.2	3.58	1.67	46.6	3.82	.87	22.8
February	3.25	2.21	68.0	3.01	2.22	73.8	3.62	1.75	48.3
March	4.69	2.28	48.6	2.63	2.92	111.0	4.41	3.91	88.6
April	3.97	5.28	131.1	2.73	3.46	127.1	1.50	3.80	253.6
May.........	2.51	5.21	208.1	2.10	2.02	96.3	4.91	4.80	97.6
June.	5.48	.79	14.4	2.79	.90	32.3	3.83	1.25	33.6
July..........	4 82	1.03	21.4	5.43	.85	15.7	4.05	.86	21.2
August	5.77	1.11	19.3	3.60	.70	19.5	3.42	.82	24.0
September...	3.37	.87	25.8	4.28	.66	15.4	2.08	.82	39.4
October......	2.22	.72	32.4	4.15	.77	18.6	4.17	.76	18.2
November ...	3 89	1.34	34.5	3.15	1.19	37.8	5.13	1.71	33.3
December....	4.24	2.49	58.7	2.57	.89	34.6	5.99	2.53	40.6
	47.24	24.91	52.6	40.02	18.25	45.6	46.93	23.88	51.0

MONTH.	1884.			1885.		
	Rain. Inches.	Flow. Inches.	Percent.	Rain. Inches.	Flow. Inches.	Percent.
January.........	3.81	1.31	33.4	4.14	3.89	93.9
February.......	4.75	3.60	75.9	2.92	1.50	51.4
March..........	4.56	3 59	78.8	1.47	1.32	89.6
April.....	2.65	6.95	252.1	2.79	2.86	102.4
May............	3.90	4.30	110.4	2.43	2.91	119.7
June	2.75	1.20	43.6	3.20	1.28	40.0
July............	5.17	.86	16.6	3.56	.89	25.0
August	4.22	.73	17.3	8 06	1.03	12.8
September......	1.80	.72	40.1	1.84	.88	47.8
October.........	3.15	.73	23.2	4.72	1.13	24.0
November..	3.56	1.16	32 6	5.20	3.60	69.3
December.......	4.83	2.15	44.4	3.29	2.36	72.0
	45.15	27.30	60.4	43.62	23.65	54.3

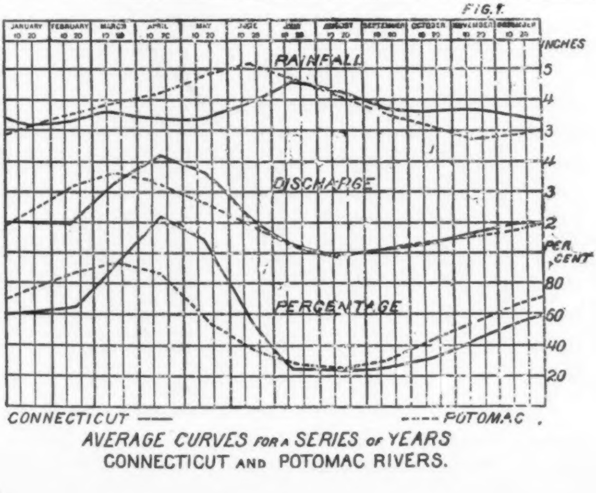
In Table No. 2 the averages for the preceding table are given: column 1, the average monthly precipitation for the entire 13 years; column 2, the average discharge, and column 3, the percentage of these averages. The next three columns give the so-called smoothed-out values for these figures, computed by the formula $\frac{1}{4}(a + 2b + c)$. Thus the value 3.35 ins. for February was obtained by $\frac{1}{4}(3.27 + 2 \times 3.10 + 3.94) = 3.35$. By this method abnormal observations are eliminated, and when the figures are treated graphically the resultant is a more characteristic curve.

CONNECTICUT BASIN.

TABLE No. 2.

Month.	Average rain.	Average flow.	Percent.	Smoothed rain.	Smoothed flow.	Smoothed percent.
	Inches.	Inches.		Inches.	Inches.	
January	3.27	1.93	59.1	3.26	1.99	61.2
February	3.10	2.04	65.8	3.35	2.00	66.8
March	3.94	3.00	76.3	3.56	3.19	90.9
April	3.26	4.73	145.0	3.41	4.16	124.6
May	3.17	4.19	132.2	3.40	3.64	111.5
June	4.00	1.46	36.5	3.99	2.04	56.2
July	4.79	1.02	21.3	4.61	1.14	25.2
August	4.87	1.06	21.8	4.39	1.01	23.5
September	3.04	.89	29.3	3.72	.99	27.2
October	3.93	1.11	28.3	3.71	1.22	32.7
November	3.93	1.76	44.8	3.80	1.67	44.6
December	3.39	2,06	60.7	3.50	1.98	56 3
Totals	44.69	25.25	56.5			

FIG. 1.



CONNECTICUT —— ---- POTOMAC

AVERAGE CURVES FOR A SERIES OF YEARS
CONNECTICUT AND POTOMAC RIVERS.

Fig. 1 is Table No. 2 treated graphically. It brings out the nature of the Connecticut discharge. April and May are the flood months, with percentages over 100. Although the maximum point of the rainfall curve is attained in July, practically the minimum points for the other two curves occur in this month. For purposes of comparison are shown the similar curves for the Potomac River as given in the author's paper on "Hydrography of the Potomac Basin," in Vol. XXVII, page 21, of the *Transactions* of the Society. The Connecticut curves are in full, and the Potomac ones in dotted, lines. Comparing the rainfall curves of the former stream with those for western Massachusetts as given by Mr. Desmond FitzGerald in his recent paper on "Rainfall and Flow of Streams," in Vol. XXVII, page 253, of the *Transactions*, a wonderful uniformity is discovered; three maximum points occur in all of the curves in March, July and November. When it is considered that the Connecticut curve is the resultant of the means of 12 stations for 13 years, this coincidence is significant and cannot be considered simply a matter of chance.

Tables Nos. 3, 4, 5, 6 and 7, for the Sudbury, Croton, Neshaminy, Potomac and Savannah Rivers respectively, are similar to Table No. 2. For the first-named stream the data were obtained from Mr. FitzGerald's paper above referred to; for the Croton, from the report of the State Geologist of New Jersey for 1890, and for the Savannah River, from computations based upon the work of the Army engineers in that basin. The Neshaminy data were obtained from the reports of the Philadelphia Water Board.

TABLE No. 3.

SUDBURY BASIN.

MONTH.	Average rain.	Average flow.	Percent.	Smoothed rain.	Smoothed flow.	Smoothed percent.
	Inches.	Inches.		Inches.	Inches.	
January	4.18	2.05	49.1	4.03	2.31	57.4
February	4.06	3.18	78.5	4.22	3.36	79 0
March	4.58	5.02	110.0	4.14	4.21	101.9
April	3.32	3.62	109.0	3.61	3.57	97.6
May	3.20	2.00	62.5	3.18	2.12	65.8
June	2.98	.87	29.2	3.24	1.02	32.5
July	3.78	.34	9.0	3.69	.53	15.1
August	4.23	.55	13.0	3.87	.48	12.3
September	3.23	.46	14.2	3.78	.63	16.2
October	4.41	1.02	23.2	4.04	1.03	25.0
November	4.11	1.62	39.4	4.09	1.55	38.7
December	3.71	1.95	52.7	3.93	1.89	48.5
Totals	45.80	22.67	49.5			

TABLE No. 4.

CROTON BASIN.

MONTH.	Average rain.	Average flow.	Percent.	Smoothed rain.	Smoothed flow.	Smoothed percent.
	Inches.	Inches.		Inches.	Inches.	
January........	3.65	2.12	58.2	3.53	2.19	62.3
February.......	3.30	2.47	74.9	3.65	2.72	74.4
March..........	4.36	3.80	89.6	3.92	3.40	87.7
April	3.64	3.51	96.5	3.73	3.32	89.2
May............	3.28	2.44	74.4	3.47	2.38	68.6
June...	3.66	1.06	29.0	3.63	1.29	36.9
July............	3.92	.60	15.3	3.82	.83	21.9
August.........	3.76	1.05	28.0	3.86	.91	23.7
September	4.00	.93	23.3	3.94	.98	25.0
October	4.00	1.01	25.3	4.00	1.07	26.8
November......	3.98	1.33	33.4	3.87	1.43	37.5
December	3.53	2.04	57.8	3.64	1.90	51.8
Totals......	45.08	22.30	49.6			

TABLE No. 5.

NESHAMINY BASIN.

MONTH.	Average rain.	Average flow.	Percent.	Smoothed rain.	Smoothed flow.	Smoothed percent.
	Inches.	Inches.		Inches.	Inches.	
January..	4.28	3.98	93.1	4.17	3.69	87.5
February.......	4.42	4.22	95.2	4.25	3.98	93.5
March...	3.88	3.51	90.5	3.81	3.36	87.0
April	3.06	2.19	71.6	3.44	2.23	65.3
May	3.75	1.03	27.6	3.47	1.50	36.1
June	4.31	.75	17.4	4.61	.98	21.2
July............	6.05	1.34	22.2	5.33	1.19	22.3
August	4.91	1.35	27.5	4.94	1.31	26 9
September	3.88	1.18	30 4	4.18	1.26	30.4
October.., ...	4.04	1.34	33.2	4.20	1.40	35.7
November......	3.84	1.73	45.1	3.88	1.84	42.8
December.	3.78	2.56	67.8	3.92	2.71	68.5
Totals......	50.20	25.18	50.1			

TABLE No. 6.

Potomac Basin.

Month.	Average rain.	Average flow.	Percent.	Smoothed rain.	Smoothed flow.	Smoothed percent.
	Inches.	Inches.		Inches.	Inches.	
January	3.21	2.09	65.2	3.09	2.21	70.4
February	3.35	3.36	100.1	3.57	3.11	87.0
March	4.39	3.62	82.6	3.90	3.53	91.8
April	3.48	3.51	101.0	4.12	3.25	82.7
May	5.11	2.36	46.3	4.74	2.54	57.6
June	5.25	1.93	36.8	5.12	1.81	35.1
July	4.89	1.00	20.5	4.71	1.18	24.6
August	3.81	.78	20.5	4.09	.91	22.3
September	3.86	1.06	27.5	3.54	1.03	30.3
October	2.65	1.21	45.7	3.01	1.32	45.3
November	2.88	1.79	62.3	2.75	1.53	55.4
December	2.59	1.32	51.1	2.82	1.63	57.4
Totals	45.47	24.03	53.0			

TABLE No. 7.

Savannah Basin.

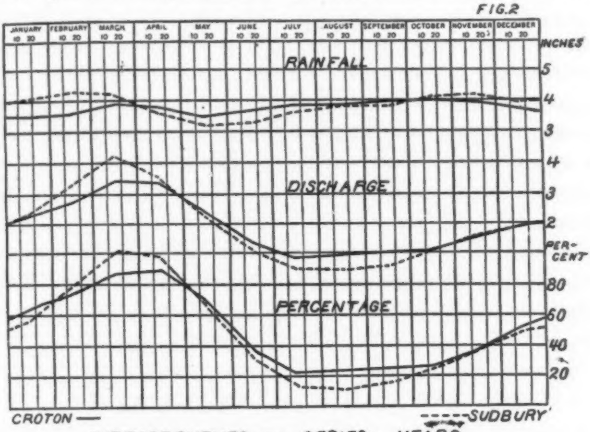
Month.	Average rain.	Average flow.	Percent.	Smoothed rain.	Smoothed flow.	Smoothed percent.
	Inches.	Inches.		Inches.	Inches.	
January	4.34	2.50	57.6	3.72	2.22	59.5
February	3.47	2.59	74.6	4.01	2.70	67.8
March	4.86	3.13	64.4	3.83	2.71	74.8
April	2.08	1.99	95.7	3.27	2.15	73.3
May	4.05	1.51	37.3	3.65	1.61	50.5
June	4.44	1.41	31.8	4.85	1.45	30.9
July	6.46	1.47	22.8	5.49	1.58	30.0
August	4.59	1.96	42.7	4.83	1.78	38.8
September	3.70	1.73	46 8	3.77	1.67	44.5
October	3.03	1.26	41.6	2.86	1.39	52.3
November	1.68	1.33	79.1	2.27	1.31	62.0
December	2.71	1.31	48.1	2.86	1.61	58.2
Totals	45.41	22.19	48.9			

In Figs. 2 and 3 these tables are also represented graphically, similar to Fig. 1. The Sudbury, Croton and Neshaminy are relatively small basins, having, respectively, 76, 353 and 139 sq. miles of drainage area; but in the form as presented in the tables, the results are comparable with the larger streams.

Daily gauge-height observations have been maintained on the Savannah River since 1875 by the Weather Bureau. Taken in conjunction with the discharge measurements made by the Engineer

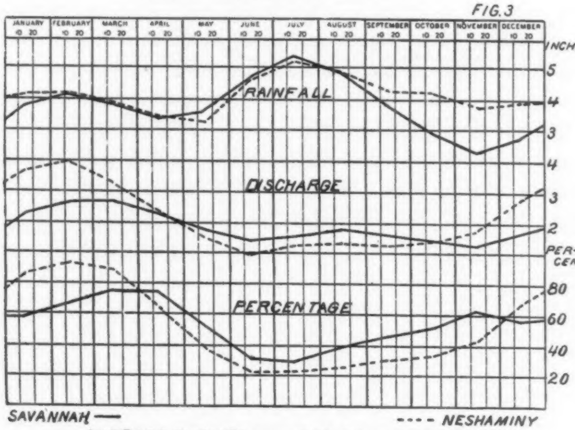
Corps, the daily discharges for this period have been computed. An inspection of the results has shown, however, that reliance can only be placed upon the computations since 1884, so that the figures for the

FIG. 2



AVERAGE CURVES FOR A SERIES OF YEARS
CROTON AND SUDBURY RIVERS.

FIG. 3



AVERAGE CURVES FOR A SERIES OF YEARS
SAVANNAH AND NESHAMINY RIVERS.

years previous to that date have been excluded from this discussion. The only rainfall data obtainable were the records for Augusta, Ga., which are taken as the average for the basin.

The Savannah River is formed by the junction of the Tugaloo and Seneca Rivers at Andersonville, Ga., and flows in a south-easterly direction to the sea. The total fall of the river between Andersonville and Augusta is 400 ft., occurring in a distance of 106 miles, and giving a grade of nearly 4 ft. to the mile. The river between Andersonville and Petersburg, a distance of 55 miles, has a fall of 288 ft., or an average of 5.25 ft. per mile. From the latter point to Augusta the slope is 2.2 ft. per mile. The drainage area of the river at Augusta is 7 294 sq. miles.

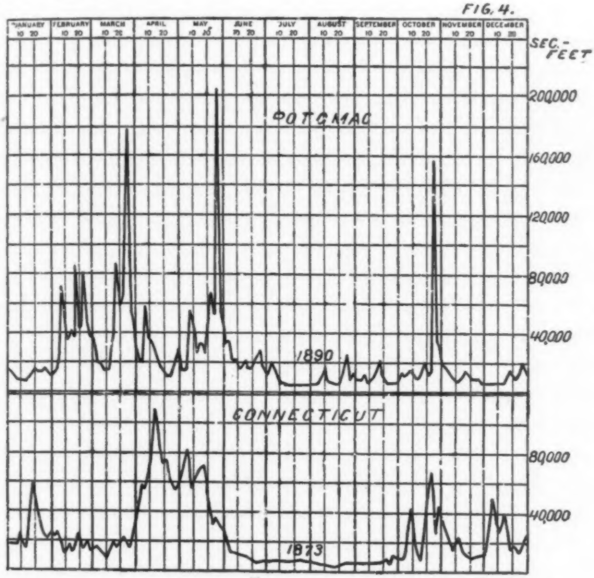
An inspection of all the above curves shows that their character is dependent upon the geographical positions of the respective rivers. The farther south the rivers are, the more uniform are their average curves. Taken, however, by individual years the reverse is the rule. The northern streams are characterized by an annual spring flood, the waters gradually rising in March or April, maintaining this high level for a few weeks and declining through June and July, with a low-water stage for the rest of the year. The southern streams, on the contrary, are subject to freshets throughout the year, rising to considerable heights and then disappearing in a day or so. These phenomena are due to two causes, climatic and topographic. The northern spring floods result from the gradual melting of the winter snows, their intensity being controlled by the characteristic topography of the northern valleys—one might almost call it an accidental feature—that is, to the occurrence of glacial drift in the entire northern section of the country. Owing to the absence of this feature in the southern valleys a rain falling upon a certain water-shed soon finds its way into the river channels. For the entire year, however, the percentage of the rainfall that finds its way into the rivers is about the same for all sections.

Fig. 4 gives the daily discharges of the Connecticut River for the year 1873, and for the Potomac River for 1890. These two diagrams are typical for their respective streams, and, in fact, are representative, one of a northern and the other of a southern river.

In Table No. 8 are given the totals for the year for the several basins under discussion. For the three smaller basins the percentages of annual discharge to precipitation are noticeable, as they vary very slightly from each other, averaging 49.7 per cent. The Savannah percentage is very near to it, 48.9 per cent. For the two larger rivers, the

Potomac and Connecticut, the percentages are larger, being respectively 53.0 and 56.5.

FIG. 4.



Daily mean discharge in cubic feet per second of the
CONNECTICUT AND POTOMAC RIVERS.

TABLE No. 8.

RIVER.	Drainage Area.	Annual Rainfall.	Annual Discharge.	Percentage.
	Square miles.	Inches.	Inches.	
Sudbury	76	45.80	22.67	49.5
Connecticut	10 234	44.69	25.25	56.5
Croton	353	45.08	22.36	49.6
Neshaminy	139	50.20	25.18	50.1
Potomac	11 043	45.47	24.03	53.0
Savannah	7 294	45.41	22.19	48.9
Madison	2 000	20.00	13.00	65.0
Missouri—Craig	17 615	14.00	4.00	28.6
Cache la Poudre	1 060	13.70	4.23	30.8
Arkansas	3 060	11.50	3.53	30.9
Rio Grande	1 400	30.70	12.84	41.9
Salt	12 260	12.00	3.50	29.2
Bear	6 000	13.00	5.40	41.5
Provo	640	.....	11.40	
Upper Snake	10 100	18.50	12.50	67.6
Owyhee	9 875	13.00	2.10	16.2
Malheur	9 900	.....	0.60	
Weiser	1 670	18.50	10.00	54.1
Carson, Nev	400	.....	24.00	

In Table No. 9 the question of run-off is treated in another way. In it the percentage of the entire discharge for the year flowing off in each month is given for a number of basins throughout the United States. The data for the western half were obtained from the work of the Irrigation Division of the United States Geological Survey in that section. The results for the Mississippi and Ohio Rivers were computed from the work of the Army engineers. They are for the year 1882 alone, but are considered not far from the average. The station for the upper Mississippi is at Grafton, Ill., just above the junction with the Missouri, with a drainage area of 164 534 sq. miles. The station on the Missouri River is at St. Charles, Mo., just above its mouth, with a drainage area of 526 500 sq. miles. The results are the means of 12 years' observations, from 1879 to 1890, inclusive. On the Ohio River the measurements were made at Paducah, Ky., where the drainage area is 200 500 sq. miles. For the lower Mississippi the station is near its mouth, at Carrollton, La., with a drainage area of 1 214 000 sq. miles.

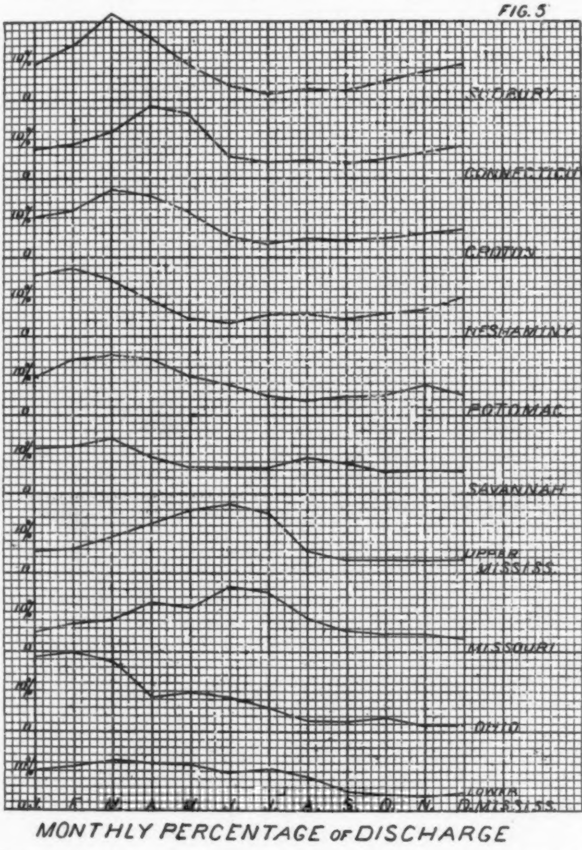
The percentages for the western streams are grouped by localities or States, and are the means for several streams for a number of years. Owing to similar meteorologic and topographic conditions, the results for the separate basins do not vary greatly from each other or from the mean for each locality. In Montana the results of work on the upper Yellowstone, Madison, Gallatin and upper Missouri are averaged; for Colorado, the Cache la Poudre, Arkansas and Rio Grande; Arizona, Salt and Gila Rivers; Nevada, Carson and Truckee; Utah, the rivers draining into Great Salt Lake; Eastern Idaho, Henry Fork Falls, Teton and Upper Snake, and Eastern Oregon, the Weiser, Malheur and Owyhee Rivers.

Table No. 9 can be used in various ways. If the average flow for a certain month for any stream is known, then from this table, by taking the percentages of the locality in which the stream occurs, the discharges for the other months and for the entire year may be obtained. Knowing the annual rainfall for a certain basin, from Table No. 8, taking the percentage of the locality in which the stream occurs, the annual discharge can be computed, then from Table No. 9, the monthly distribution of this annual discharge can be found.

It is considered for several reasons that more accurate results can be obtained by this method of computation than from figuring dis-

charges from records of monthly rainfall. To begin with, there is no uniformity in the percentages of monthly flow, as is shown by the inspection of Table No. 1. On the other hand, the yearly percentages of run-off, as given in Table No. 8, have not a great variation in range, nor have the percentages of monthly discharges, as given in Table No. 9.

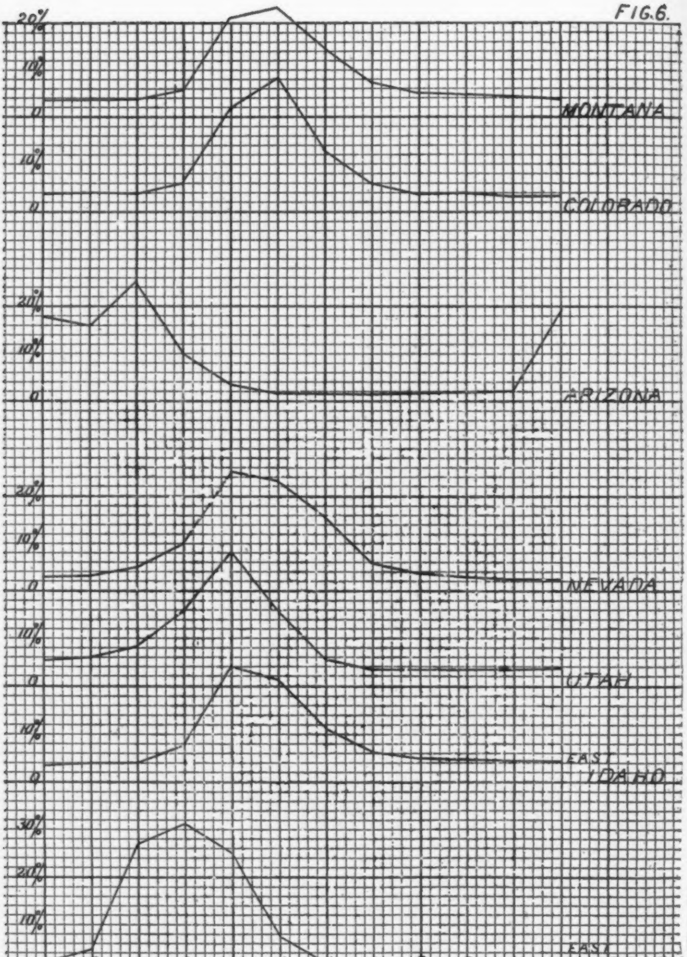
FIG. 5



MONTHLY PERCENTAGE OF DISCHARGE

The rainfall records for the western States are very unsatisfactory; little data for precipitation in the mountains are in existence, as most of the rainfall stations are out upon the plains or in the valleys. In some cases, in Table No. 8, blanks had to be left in the rainfall column.

FIG. 6.



MONTHLY PERCENTAGE OF DISCHARGE

Table No. 9 is represented graphically in Figs. 5 and 6. They also bring out the characteristics of the average curves for rivers of different sections ; that is, the curves for the southern rivers are of a more uniform character than those for the northern ones.

TABLE No. 9.

Monthly Percentages of Flow.

Basin.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
Sudbury	9.0	14.0	22.2	16.0	8.8	3.8	1.5	2 4	2.0	4.5	7.2	8.6
Connecticut	7.6	8.1	11.9	18.7	16.6	5.8	4.0	4.2	3.5	4.4	7.0	8.2
Croton	9.5	11.0	17.0	15.7	10.9	4.7	2.7	4.7	4.2	4.5	6.0	9.1
Neshaminy	15.8	16.8	13.9	8.7	4.1	3.0	5.3	5.4	4.7	5.3	6.9	10.1
Potomac	8.7	14.0	15.1	14.6	9.8	8.0	4.2	3.2	4.4	5.0	7.5	5.5
Savannah	11.3	11.7	14 0	9.0	6.8	6.4	6.6	8.8	7.8	5.7	6.0	5.9
Miss., upper	5.7	6.0	9.2	12.4	15.8	16.9	15.0	5.5	3.6	3.4	3.4	3.1
Missouri	4.3	6.5	8.5	12.1	11.4	16.3	14.7	7.7	5.6	4.7	4.7	3.5
Ohio	18.9	19.9	17.7	8.4	9.7	8.1	5.8	2.4	2.7	3.5	1.4	1.5
Miss., lower	10.0	10.5	12.7	11.4	11.1	9.1	10.5	8.4	4.2	4.0	3.7	4.4
Montana	3.2	3.4	3.6	5.7	21.7	24.6	14.3	6.4	4.9	4.4	4.1	3.7
Colorado	3.9	4.0	3.9	5.9	21.7	28.4	12.7	6.0	3.4	3.9	3.1	3.1
Arizona	17.7	15.7	25.1	10.3	3.2	1.5	1.6	1.2	1.5	1.2	2.1	18.9
Nevada	2.7	2.7	4.6	10.1	24.7	22.9	15.3	5.3	3.3	3.0	2.7	2.7
Utah	4.6	4.9	8.2	15.8	28.4	15.0	5.8	3.6	3.1	3.4	3.3	4.0
East Idaho	3.9	4.0	4.0	7.7	24.0	20.9	11.0	6.1	4.9	4.6	4.5	4.4
East Oregon	1.4	4.3	26.9	30.1	24.7	6.4	1.7	0.6	0.5	0.7	1 2	1.5

DISCUSSION.

Charles Macdonald, M. Am. Soc. C. E.—The illustration just exhibited calls to mind an experience which an honored President of this Society had, in discussing a similar question for the Ohio River.

A very distinguished engineer had proposed a means of pounding the excessive rainfall in the headwaters of the Ohio, with a view to maintaining a regular flow in that river. He discussed the question entirely upon theoretical grounds, and came to the conclusion that all that would be necessary would be to construct six reservoirs on the tributaries of the Ohio, in order to maintain a uniform flow of not less than 5 ft. in that river.

The pamphlet in which his views were presented was so well written that a very strong pressure was brought to bear upon Congress to have an appropriation made for a series of surveys taking in the headwaters of the Ohio, with a view to determining the location of the suppositious reservoirs.

Mr. Roberts took an ordinary map of the country, and in the course of a few hours' calculation of the number of square miles of rain area available for every conceivable tributary of the river, showed conclusively that there was not a single place where it would be possible to erect one of the six imaginary dams; and then he went on to prove that even supposing it were possible to find suitable locations for the six reservoirs, the so-called improvement would do more harm than good, owing to this very irregularity of rainfall.

He referred to records taken at Wheeling for 30 years previous, from which it was evident that storage reservoirs, such as had been proposed, would have, at many times, actually increased the floods in the main stream.

This is a mere outline of the statements made by Mr. Roberts in the admirable pamphlet which he wrote upon the subject. It had the effect of checking the movement which had in view the spending of a large sum of money for a purpose which was not warranted by the known facts.

William R. Hutton, M. Am. Soc. C. E.—May I say, Mr. Chairman, that Mr. Roberts' conclusions were not universally accepted. Mr. Elwood Morris and others at that time differed from him very much, and Mr. Ellet, in his report on the improvement of the Kanawha River, describes one reservoir quite as large as any he had recommended.

Mr. Macdonald.—If there is any other subject connected with the flow of water in rivers, it is a very proper time to take it up, especially as our western rivers are now said to be overflowing their banks. Perhaps some gentleman has an opinion on the advisability of spending United States money for keeping the Mississippi River within its banks.

Mr. Hutton.—May I say in reference to the subject mentioned before, that the Government has begun a number of large reservoirs on the upper Mississippi, to supply water to the river for navigation during the dry season. Some of them are as large or larger than those proposed by Mr Morris. They also, to a great extent, control the flood water.

A. Fteley, M. Am. Soc. C. E.—Having just arrived, I hardly know the subject of the present discussion, but it may not be out of place to state that the water-shed of the Croton River furnishes a very good illustration of the wide differences which are often observed in the rainfall of contiguous drainage areas. The records show that those differences are frequently very large, amounting sometimes to more than 100% in tributary valleys a small distance apart; the differences between the average rainfall on the Croton watershed and the precipitation in New York are also conspicuous.

A short time ago this condition of facts was well illustrated in connection with the construction of one of the dams now being built by

the Aqueduct Commissioners on a small tributary of the Croton River. Owing to special local circumstances, a temporary dam with an earth canal and timber flume had been erected, to divert the stream over the foundation works; the size of the temporary structure was very large as compared with the conditions generally obtaining in the region, but the excessive rainfall which took place in that particular valley at the time caused an exceptionally high freshet which taxed the temporary channels just mentioned much beyond their capacity and caused some damage.

The tables of rainfall on the Croton water-shed have been published and are well-known, and they do not show very wide differences from year to year.

E. B. DORSEY, M. Am. Soc. C. E.—What is the flow at the Croton Dam as compared to the total rainfall on the water-shed ?

Mr. FTELEY.—The average flow, I should say, 49 per cent.

Mr. WORTHEN.—Thirty-five; around there.

Mr. DORSEY.—Taking a series of years ?

Mr. FTELEY.—Taking a series of years it is 49, almost exactly.

Mr. MACDONALD.—In addition to the amount consumed by the city ?

Mr. FTELEY.—No, sir; we refer to the percentage of water which flows in the stream, as compared with the total of rainfall over the whole water-shed.

Mr. DORSEY.—Can any of that 49% escape by flowing under deep sand or through the gravel ?

Mr. FTELEY.—Hardly so, because the rock is very near the surface, and there are on both sides of the valley some pretty high and rocky hills. I hardly suppose there is any considerable quantity that can disappear in that way. In a very dry season, when all the flow of the river is retained by the storage reservoirs, the flow in the river below Croton Dam is insignificant.

JOHN BOGART, M. Am. Soc. C. E.—Mr. President, it seems proper that I should say in regard to this paper that, in my opinion, the hesitation in discussing it here this evening is not due to any lack of comprehensive statement in the paper, but to the fact that the subject is one of great delicacy and intricacy, and that it would be almost impossible to properly discuss a paper which is as condensed as this, with a very large amount of information shown in a tabulated form and by diagrams, without a careful comparison of this paper with other data and information which can only be found in an office and with considerable study. Mr. Babb, however, the writer of the paper, makes some decided statements which are in certain respects different from conclusions that have been arrived at by other engineers who have given a great deal of attention to this subject. For instance, he claims, on page 12, that his Table No. 9 can be used in various ways. "If

the average flow for a certain month for any stream is known, then from this table, by taking the percentages of the locality in which the stream occurs, the discharges for the other months and for the entire year may be obtained." That is a pretty large claim for this table. He further says: "Knowing the annual rainfall for a certain basin, from Table No. 8, taking the percentage of the locality in which the stream occurs, the annual discharge can be computed; then from Table No. 9, the monthly distribution of this annual discharge can be found." That is another pretty large claim. I am inclined to think that the careful judgment of an experienced engineer had better be applied to a study of all the records available in each case, and that the conclusions resulting from such a study are more reliable than arbitrary tabulations.

H. M. MARSHALL, M. Am. Soc. C. E.—To one who has plotted gauge heights as ordinates and measured discharges as abscissas, for an extended period of observation, it is well known that the resulting curve forms a loop near the maximum, and that the maximum discharge exceeds by quite a considerable amount the discharge at the time of the highest gauge reading. An example is shown in the January number of *Ponts et Chaussées*, in the article by M. Voisin Bey, on the improvement of the mouth of the Danube. It follows from this that a relationship discovered between a gauge and the discharge, if founded on a few observations, is apt to be illegitimate. To show how wide of the mark one will go in guessing discharge from gauge reading, reference is made to the Bulletin of the Weather Bureau, giving gauge heights of the Mississippi and its principal tributaries for 1890–92. On page 7, the discharge of the Red River at Alexandria, La., is stated to be 2 500 cu. ft. per second for a stage 2.2 ft. The measured discharge there was 4 826 ft., when the gauge read 1.7 ft. The discharge measured at the next higher stage was 8 613 cu. ft. per second, with the gauge + 1.9 ft. Which goes to show, we may again be discussing why a fish weighs more when alive than dead; and as Mr. Babb has served his fish in a tempting dish, we had best look into his cooking before we swallow.

Table No. 9 shows a small range of percentage, but being percentages of large or relatively large quantities, a small variation means more than it looks.

For instance, the lower Mississippi minimum discharge is given by that table 3.7%, and the maximum 12.7 per cent. The Mississippi River Commission report, 1883, page 236, gives for the former 240 000 cu. ft. per second at Red River Landing, and for the latter, 1 400 000. A variation of 1% in Table No. 1 would therefore correspond to a difference of about 100 000 cu. ft. per second, or from one-fourth to more than one-third of the discharge for a month.

The discharge for the Connecticut for January is given at 7.6% of

the total discharge; but the average outflow for 13 years, which, by the way, is derived by dividing 12 years' observations by 13, is stated in Table No. 2 to be 1.93 ins. In the absence of exact information as to how the 7.6% was arrived at, it is fair to suppose it was derived from Table No. 2, and corresponds to the 1.93 ins., which is 7.6% of the total 25.25 ins. Then, what would be the error if the outflow for January, 1874, should be calculated on that basis?

It matters little how the percentages are derived, whether from a single year or the average of a number of years, they appear as constants for each stream and for streams in the same region; hence they cannot be a measure of the outflow, which is variable from year to year and month to month. That the outflow has as wide range of variation as the rainfall is seen by comparing the maximum 30.81 ins. of 1874 and the minimum 18.25 ins. of 1880, with the rainfall 50.20 of 1878 and 40.02 of 1880. In view of the claim that "the yearly percentages of run-off, as given in Table No. 8, have not a great variation in range," it is somewhat astounding to find 65.0 for Madison and 28.6 for Missouri Craig, in the last column of that table in sequence, and for rivers of the same region at that.

Because the outflow may be least when the rainfall is greatest, or *vice versa*, can hardly be taken as proof that the former is not an implicit function of the latter. The complexity of the problem is well stated by Mr. F. H. Newell, in the discussion on Hydrography of the Potomac Basin, and under the title, Influence of Forests on Water Supplies, in the report of the Department of Agriculture for 1889, page 297 *et seq.*, Mr. B. E. Fernow, Chief of the Forestry Division, writes on the subject in a manner as entertaining as it is instructive. That the matter lacks much of being reduced to a scientific basis goes without saying, but that the truth is to be sooner reached by discarding the only usually known cause is incredible. Furthermore, while the results of experiment and observations are the touchstone of theory, they can scarcely originate more than empirical rules which fit only like cases. It is far easier to reason from causes to combined effect than to assign a law from effects.

In regard to the "smoothed rainfall" and "smoothed flows," it is difficult to understand how they are more characteristic after their distinguishing features are taken out of them. The figures were not only reduced by averaging for all the same months, but they must needs be made like all other months.

He who sees only defects without offering a remedy is only a scold; therefore suppose

$$y = ax + bx_1 + cx_2, \text{ etc.},$$

where y is the outflow for a month and x, x_1, x_2, etc., are the rainfall for that month and the preceding months, respectively, and a, b, c, etc., are coefficients to be determined for each month from observation. By

inserting the values of y and the x's from a series of observations, equations of condition will be formed from which a, b, c, etc., can be evaluated. The differences between computed and observed values of y, when the right-hand member of the equation is extended or contracted by one month, will determine when the rainfall for a preceding month ceases to be significant, the form which gives the least residuals being accepted for that month.

If this method seems complicated, it is no more so than the problem, and in that wise ought to be a hit.

CLEMENS HERSCHEL, M. Am. Soc. C. E.—The tendency of mankind to follow routine and precedent, has often been noted by philosophers, and its tenacious hold upon men may be observed in their several walks of life. Nothing else will probably explain the insistence of engineers, when about to portray the flow of rivers, first, to sheer off into a portrayal of rainfall, and pass by a clumsy, unscientific, untruthful system of monthly proportions back to the desired result.

The direct method would seem to be simple enough. If we are interested in and are plotting monthly averages, let us plot for each month the maximum monthly average, the minimum monthly average, and, if we like, the average monthly average for all the months of the years of gauging. We thus obtain a picture of the range to be expected in each month for that river, and for similar rivers in the same section of country; and no juggling with the data thus represented can ever teach any more than these primary data. A diagram of this kind, by the writer, may be found in the volume for 1878 of these *Transactions*.

If we are interested in daily amounts of flow, an excellent way to represent them graphically is to arrange the 365 daily discharges that go to make up a year's flow, in their order of size. Several years' flow, thus represented, will again give breadth to the line showing the year's discharge, and this breadth will indicate the range that may be expected in a year's flow. No observations of rainfall dragged into this study can ever teach any more about the river flow than such data will teach; so why consider both simultaneously?

Rainfall, again, forms a study by itself. Engineers are interested sometimes in annual rainfalls; more frequently, only in maximum rainfalls during short periods of time.

To express by a percentage the relation between a certain flow, or run-off, during one period of time, and the rainfall during another, may, under certain circumstances, be expressing a relation of cause and effect; and again, may, as it not infrequently does, be the forced linking together of two quantities, only remotely or partially dependent the one upon the other. When thus joined in discussing the flow of rivers, this relationship has, to be sure, a passing, purely scientific, interest, but one that cannot be put to any useful purpose. In the

case of engineers, it only detracts from a proper study to be given to river-gaugings, which alone form the basis of engineers' knowledge of the flow of rivers, and, unaided and alone, will better do the work required of them by the engineer than if clouded and obscured by an uncalled-for admixtnre with considerations of rainfall on the same territory, and of a series of untruthful and therefore worthless coefficients of percentage.

Suppose we consider the discharge of the Mississippi River past New Orleans, on any one day. Will anyone pretend that that body of water, some of which has been months coming thousands of miles, can properly be made to bear any relation to the rain that fell that day over all that continental drainage area? Does it help matters any to compare the flow in March to the rainfall in February? Isn't one as untruthful as the other, and neither of any use?

Or take the small drainage areas that figure in water-works reports. Suppose no rain fell in a certain month, which has happened and is not unlikely to happen again. The rivers would not all dry up, and for any flow, we should have the proportion of rainfall yielded that month to equal an infinite number of 100% of rainfall. Something like this we have annually recurring, when March is made to yield 180 and 150% of rainfall, due to snow that fell in February flowing off in March; and other months are similarly maltreated. But the presence of snow only serves to show, in a glaring manner, a delay in run-off that takes place constantly, either due to distance to be traversed and depending in amount largely on the slope of the territory to be traversed; or due to the rain soaking into the ground and appearing many months after, may be in the form of springs in the channel of the river itself.

Here is a system, then, which is of no use, and is so hideously unscientific and untruthful as to be the reverse of ornamental, and which should, therefore, cease to form the model upon which reports on river-flow are fashioned.

If any comparison between rainfall and river-flow is permissible, it is only that of the annual measure of each. And, even then, care should be taken so to choose the beginning and ending of the year cited that no large credits or debits exist when the accounts open, and again when they close. For this purpose, the 1st of January, as generally taken, is a singularly infelicitous date. Some years rivers in northern latitudes are then in a drought, although they may then have a large volume of the last year's rainfall lying upon their drainage area. Other years, they may then be in a freshet, with the ground bare and frozen hard, and no rain for a month past to speak of. The fairest time to begin and end such an annual comparison of rainfall with river discharge would probably be during the summer months, say, June 1st or July 1st.

Reference has hitherto been made to but two factors in the formula which is to express the relation of rainfall to river-flow, namely, drainage area and amount of rainfall. Observations show, however, that the distribution of rainfall, the times and the rates at which it falls, have a most potent effect on this relation. Hence it comes that years of practically the same rainfall have widely differing amounts of run-off, not to speak of the differences of yield of months and of single showers, having the same measure in inches of rain.

The slopes of the drainage area, and the nature of the surface, are additional factors to be taken into account. Thus, on the same river, the Durance, in France, from a seven years' record, the upper portion yielded 74% of the rainfall, while a dividing line drawn further down stream, and although it included the upper portion just named, yielded only 67% of the rainfall on it during the same period of time.

Thus, also, the same eight years' records on the Perkiomen, Neshaminy and Tohickon, of 152, 139.3 and 102.2 sq. miles, respectively, of adjoining drainage areas, yielded 52.4, 48.8 and 60% of the rainfall, respectively.

Thus, also, the Pequannock drainage area, down to Charlottsburgh, from June 1st, 1891, to June 1st, 1892, yielded 23.23 ins. of run-off, while the drainage areas of the Croton and of the Sudbury, all three being situated in the same range of rainfall, and having, as far as known, practically the same rainfall and distribution of rainfall during that time, yielded only 15.74 and 15.63 ins. of run-off, respectively.

It is in the study of the effect of these additional factors upon the run-off of streams, in the study of the effects of slopes, of quality of surface, and, to some extent, of the distribution of rainfall, that the customary percentage proportions of rainfall to flow of streams have their proper places, and have value. But not as expressing, or for the purpose of foretelling what the flow of the river that has been observed will be at other times, or for the purpose of estimating what the flow of similar rivers in the same region of country is likely to be. For these latter purposes, it should stand to reason that no artificially complicated expression of the observed or gauged flow of a river will ever teach more than will the same results expressed in the standard cubic feet per second, or 1 000 000 galls. in 24 hours. To guess at half, and multiply by two, produces no better result than to guess at the whole. And to call a thing 50% of 4 is no nearer truth, and teaches no more, than to call it plain 2; while it is, on the other hand, a good deal of deception, and is a sham.

F. H. Newell.—The above paper prepared by Mr. Babb brings together the results of studies made by him of all of the available data concerning the Connecticut and Savannah Rivers, and has been presented with facts relating to other streams of the United States. The matter is given in the most concise form, as it would obviously be im-

possible to present the details; but it serves to call attention to the fact that these are to be had for future study. In considering any particular problem the engineer should undoubtedly go to the original records, in order that he may judge for himself as to the value of the deductions and their applicability to the case in hand. Papers such as the above have their value as showing general conditions and tendencies; but the averages and percentages presented in the tables should not be used as though relating to established facts. At best they represent conditions which have prevailed, and which may continue to do so.

In connection with the data presented by Mr. Babb, he brings out fairly well the fact that the relation between the measured rainfall and the measured discharge of a river is so remote or uncertain that computations based upon this assumed relation have a dubious value. In concluding he suggests that the run-off may be estimated in another manner, one in which the question of rainfall does not enter. His statement of this, however, was not sufficiently guarded and reads as though definite results might be obtained, although, as is obvious, computations of discharge made by any method are little more than approximations, since each of the factors represents strictly, not what is, but what has been. His table No. 9, if used with this fact in mind, becomes, at times, very convenient in estimates of the probable water supply at various points. At all times, however, the engineer must bear in mind that most streams are erratic, and because a certain routine has been followed in past years there is no guarantee that the same order of things will prevail at any particular time. As stated in a preceding discussion, the non-periodic oscillations of river flow, depending upon slight temporary changes of climate, are exceedingly great, and cannot be correlated with any one factor, such as rainfall.

The paper shows how little, rather than how much, information is at present available concerning the behavior of the rivers of the country. The data are exceedingly fragmentary, and in discussions of floods and droughts it is necessary to depend upon testimony of the most illusive character, viz., the memory of the "oldest inhabitant." It is generally conceded that the rivers of the Atlantic slope at least have been changed in their regimen by the progress of improvements at their headwaters. In most cases this idea seems to rest upon general statements that 40 or 50 years ago the river was never as dry as at present, or on similar assertions. In examining any long record of river flow, of which few are available, it is difficult to detect any progressive changes continuing for more than two or three years at a time, and yet, at the same time, there are sufficient variations and of a character so decided as to make a lasting impression upon the memory of an observer. Thus, in the absence of measurements it is possible for

various divergent theories concerning the behavior of rivers to find support from the testimony of living witnesses.

WM. E. WORTHEN, Past-President.—The paper presented contains a great deal of valuable information, and there is often a necessity of taking averages when nothing else can be be had, "and smoothing out of values" enables one to make good-looking average curves, but not practical value in comparison with the actual facts as given by him in Fig. 4. It is better to compare the absolute flows of streams with each other rather than by averages of a few months, or even years, of a single stream. We must, if possible, use such figures, and I have made a diagram of the daily flows of the Connecticut River, from the reports of the Engineers Corps U. S. A., and submit one of the Croton flows from 1881 to 1892, inclusive, from the data furnished by the Department of Public Works.

The ordinates of flows are on a scale of 100 000 000 galls. The space mostly in white at the bottom of the profile presents the supply to the city from 1880 to July, 1890, 80 000 000 to 100 000 000 galls.; from July, 1890, out, 150 000 000.

The ordinates within this space represent quantities drawn from reservoirs when the natural flows were insufficient.

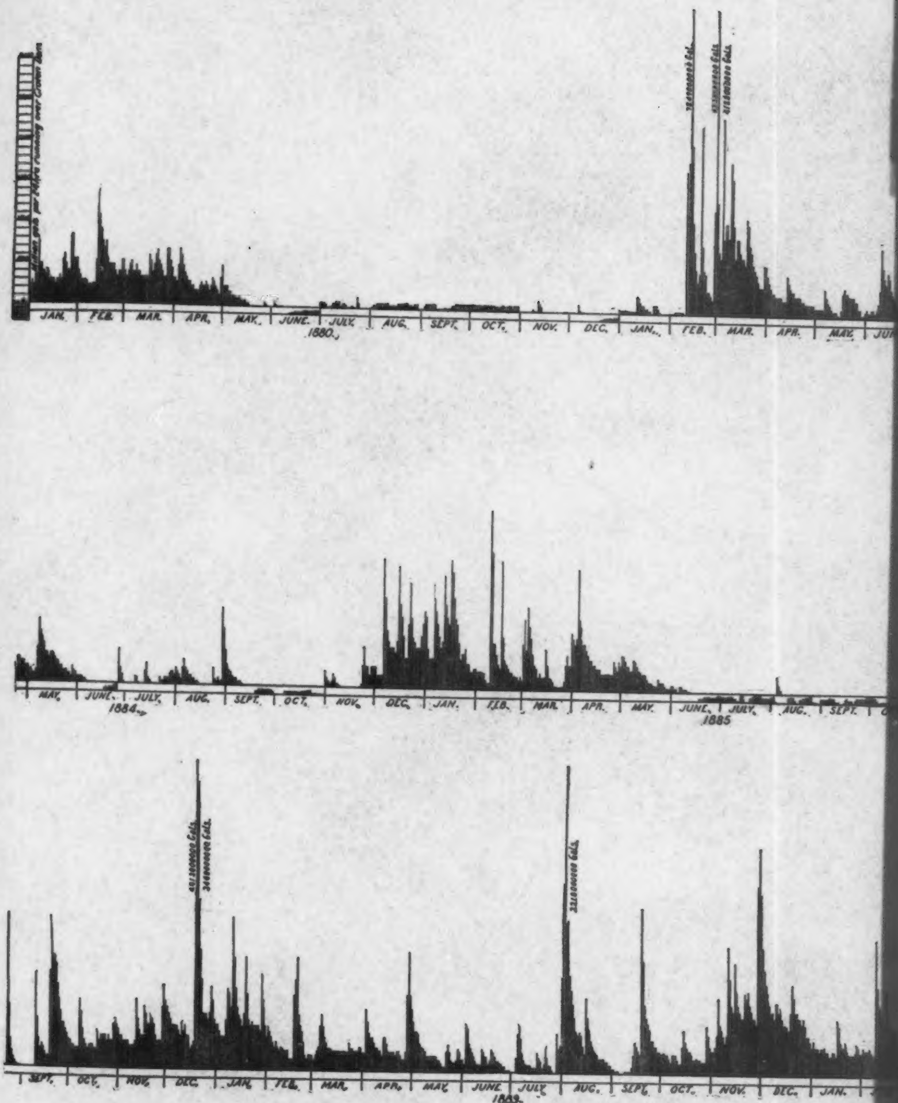
Such profiles enables an estimate to be made of the practical value of the stream for city and town supplies, or for use as water-power.

It is evident, by this profile, that without reservoirs the Croton flows would fall short of uniformly supplying the City of New York. To estimate how much could be drawn if pounded, and the capacity of reservoirs needed, try, for example, a daily supply of 300 000 000 galls., draw a line through the ordinate of this flow, which will be parallel to the base; the ordinate above the line will represent the flows which are now washed over the dam, and the capacity of the reservoirs to secure this waste by storage.

By trials of lines of different daily supplies, the capacity of reservoirs may be determined and possibilities of such reservoirs from a contoured map of the shed.

It will be found that extreme freshets often cannot be ponded, and that at these times there must be waste. With large water area of reservoirs the evaporation becomes a large factor, and loss by leakage is also to be estimated. The usual practice is to take the water area of reservoirs from that of the shed on the hypothesis that the rain water falling upon the pond would compensate for the evaporation.

In constructing this table of flows, the ordinates are taken at $h^{\frac{3}{2}}$, Formula $A = Cl \times h^{\frac{3}{2}}$ C and l being constant, the scale of quantities can readily be constructed, and calculations are avoided. If at any time C can be absolutely determined from the flows over the dam, no new profile is involved, but merely a change of scale.


JAN. FEB. MAR. APR. MAY. JUNE. JULY. AUG. SEPT. OCT. NOV. DEC. JAN. FEB. MAR. APR. MAY.
1880.
MAY. JUNE. JULY. AUG. SEPT. OCT. NOV. DEC. JAN. FEB. MAR. APR. MAY. JUNE. JULY. AUG. SEPT.
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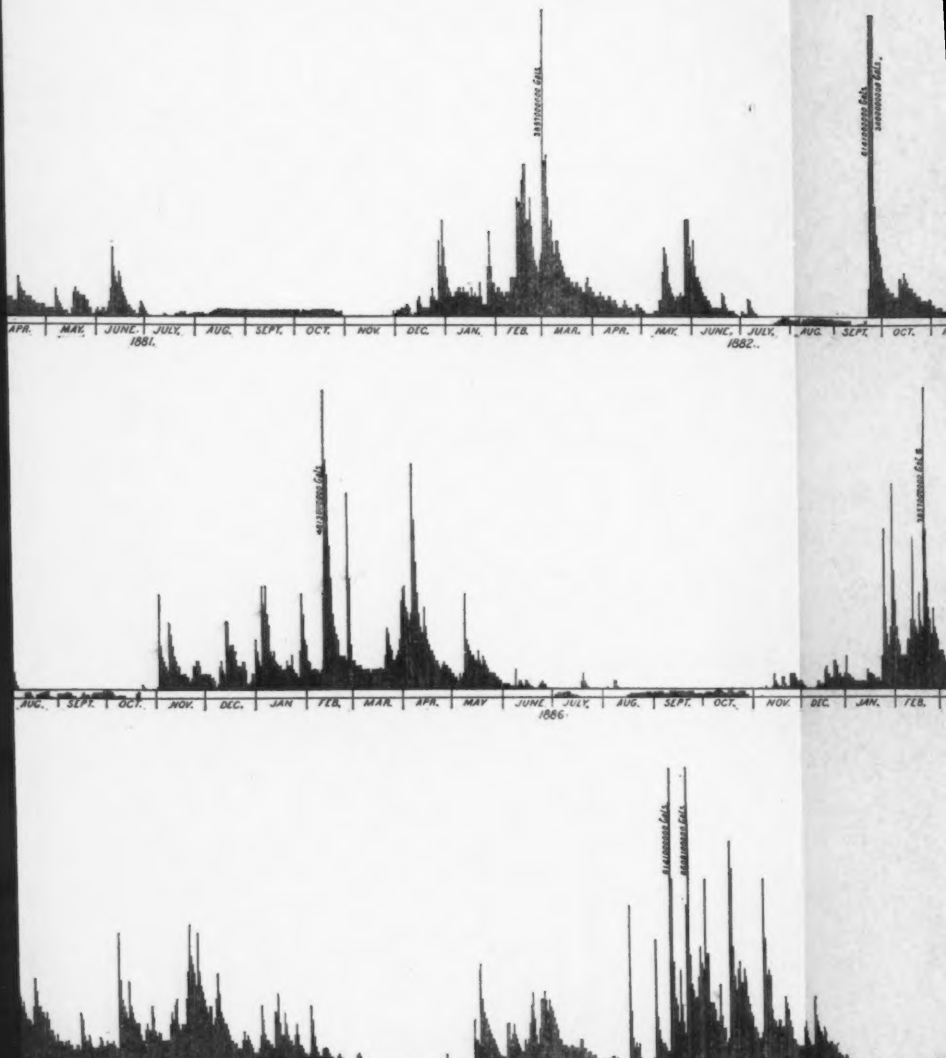

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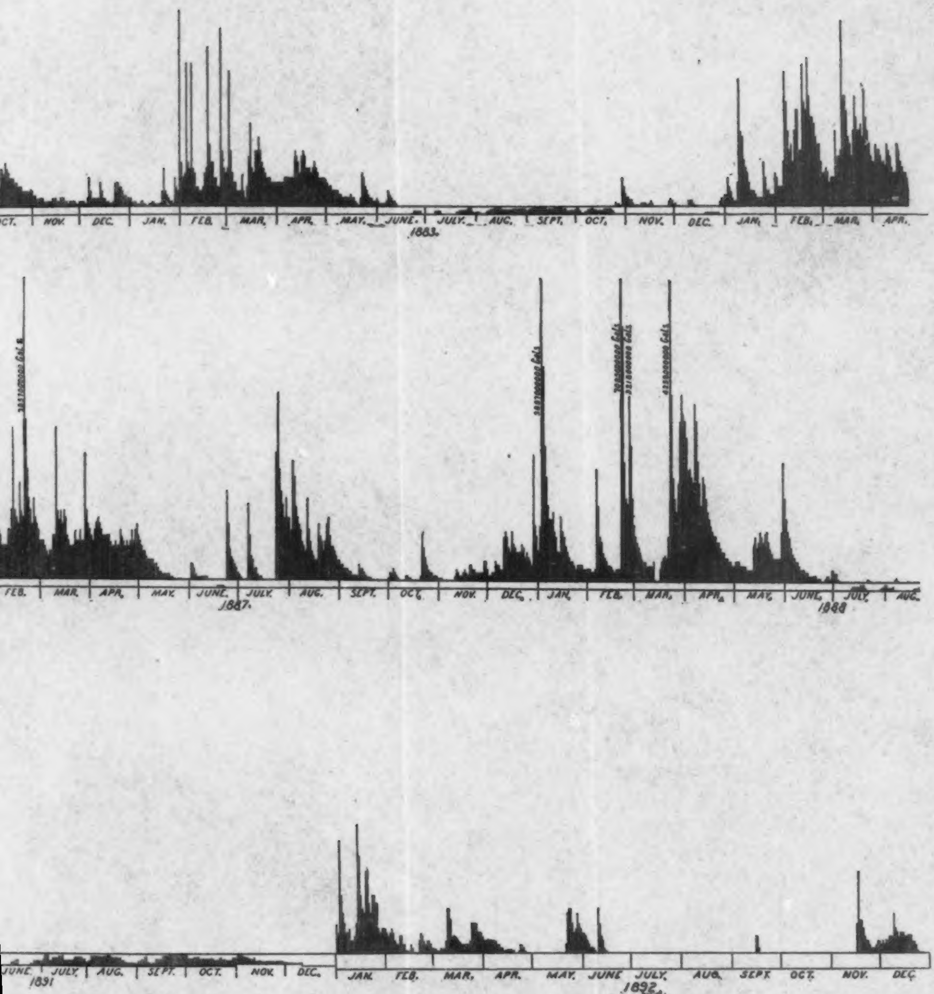
PLATE XLIX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXVIII, No. 594.
BABB ON RAINFALL AND RIVER-FLOW.



I have found this profile of the Croton of value in determining the amount of water power diverted by the several dams.

Consider the flows of the Croton as applicable to any water-shed upon it. The Croton shed is 354 sq. miles; now, if the partial shed on which the water is diverted is 17 sq. miles, then it is only necessary to enlarge the scale of the full shed on the proportion of 354 to 17, and the daily flows of the partial shed can be at once measured fairly to the proprietors of the small shed; since the large flows would be proportionally greater and the small smaller than on the full shed, excessive flows cannot be utilized as the wheels are drowned, neither can very small flows, because no power can be utilized from the wheel.

It is shown by the paper that there are many streams in which the daily flows have been kept, and it is of more practical value to have the profiles similar to those that I have given of the Croton than any system of averages from month to month. The variations from year to year are as important as the monthly or daily changes, and it is of more importance to manufacturing industries and municipal supplies that records should be kept of the flow of streams than of rainfall.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
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NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

595.
(Vol. XXVIII.—May, 1893.)

THE CONSTRUCTION OF A WATER-TIGHT MASONRY DAM.—DISCUSSION ON PAPER No. 585.

By D. FitzGerald, T. C. Clarke, S. B. Russell, E. S. Gould and W. McCulloh.

Desmond FitzGerald, M. Am. Soc. C. E.—It must be a source of satisfaction to the engineers in charge of the Croton system, even if the city itself does not appreciate the fact, that they have been able to secure a dam of the height described in the paper which is so perfectly water-tight. Almost all of the dams which I have been able to examine leak more or less, and especially where they approximate the height of the Sodom Dam. Dam No. 4 of the Boston water works, which is an embankment with a concrete core, leaked at the rate of 175 000 galls. daily when it was first completed. It is very nearly as high as the Sodom Dam, but is over ¼ mile long. From some careful observations made under my direction I am satisfied that these leaks are gradually silting up. The leakage now is not far from 100 000 galls. daily.

This dam was designed and built by Mr. Fteley. It is very necessary to exercise care in separating surface water and water really coming from the water tables below a dam from the true leakage, and it

requires a long series of observations through different seasons and under the varying conditions of rainfall to do this intelligently. I am now building, under the direction of the City Engineer, William Jackson, M. Am. Soc. C. E., another dam of the same dimensions as No. 4, and special arrangements will be made to measure the leakage. As long as leakage takes place at very low velocities and without carrying along with it the finer portions of the soil, no damage can result, and a gradual silting up must take place. It is certainly, however, a matter of congratulation to be able to build a perfectly tight dam.

T. C. Clarke, M. Am. Soc. C. E.—The experiments upon strength of mortars made of Portland and American rock cements, extending as they do over four years, are of the greatest value to engineers.

These experiments show that at the end of three years a mortar made of 1 part of Portland cement to 3 parts of sand has a strength of 572 lbs. per square inch, and a mortar made of 1 part of American rock cement to 2 parts of sand 514 lbs. per square inch, which approach so nearly as to be practically alike. After several more years they will probably coincide, as we know that rock cements are slower acting throughout.

An interesting fact, which I have never seen mentioned, is that the weights of cement are nearly the same in both cases.

If, instead of describing them as 1 to 2 and 1 to 3, we make sand the unit, we shall have—sand, 1; Portland cement, $\frac{1}{4}$; sand, 1; rock cement, $\frac{1}{3}$. If we have barrels as units—sand, 1 bbl.; rock cement, $\frac{1}{3}$ of 300 lbs. = 100 lbs.; sand, 1 bbl.; Portland cement, $\frac{1}{4}$ of 386 lbs. = 96 lbs., showing that nearly equivalent strength is due to nearly equivalent weights of cement.

The inference to be drawn is that engineers should specify use of cement by weight instead of by measure.

To make mortars of equal strength, we should specify:

Portland—$1\frac{1}{5}$ bbls. or 5 bags of 93 lbs. = 465 lbs. of Portland cement to $3\frac{3}{5}$ bbls. of sand.

American rock—$1\frac{3}{5}$ bbls. or 5 bags of 96 lbs. = 480 lbs. of rock cement to $3\frac{1}{5}$ bbls. of sand.

To make a stronger or weaker mortar, increase or diminish weight of cement (not bulk) in same ratio.

To make a reliable concrete the bulk of broken stone or screened

gravel should never be more than double the bulk of sand, as otherwise the mortar will not coat all the particles.

Economy should rather be sought by embedding rough stones in the mass of concrete, and if spaces of at least 6 ins. are left between the stones all around, the mixture will make as strong a concrete as if it were all of small stones and mortar.

The advantages of specifying by weight instead of bulk are that you are not obliged to take specific gravity, or weigh a strict bushel of cement, and you always get the quantity you pay for. All the inspector has to do is to weigh enough bags or barrels to be sure he gets the weight invoiced.

S. Bent Russell, M. Am. Soc. C. E.—From my experience with water-tight masonry construction, I would judge that a large part of the success attained in the Sodom Dam is due to a rich mortar of Portland cement. For water-tight work the mortar should never be poorer than 1 to 2. In small batches made in the laboratory 1 volume of cement to 3 volumes of sand will give a solid mixture. In contract work, however, the mixing is never so thorough as to give a mortar of uniform richness throughout; hence, to insure all voids being filled, a proportion of 1 to 2 should be insisted on, and in some cases even more cement should be used.

Mr. McCulloh makes no mention of cracks in the masonry, so I infer that there are none. Our experience on the St. Louis water works extension indicates that a masonry wall over 400 ft. long is almost sure to crack in cold weather. The crack usually follows the joints, but occasionally a large stone will be pulled in two. The crack usually closes together again at the return of warm weather. Brick masonry acts the same way, the cracks showing about the same amount of contraction, and occurring at about the same interval in length.

In laying water-tight stone walls I have had the least success at joints when new work is laid upon old, or when some time has elapsed between the laying of the course and the bedding of the stones which rest upon it. Water will seep through such joints in the majority of cases, even when the stone has been swept and cleaned with some care.

E. Sherman Gould, M. Am. Soc. C. E.—I have just read with much pleasure the advance proof of Mr. McCulloh's paper. The absence of the illustrative drawings makes it impossible to follow the descriptions as satisfactorily as if they were at hand, and for this

and other reasons, I do not discuss the paper in detail. But I cannot let the opportunity pass by of congratulating all concerned upon the very successful results obtained, and of giving my hearty assent to Mr. McCulloh's summary of the causes which in his opinion contributed to secure these results. It is greatly to be desired that all engineers in charge of whatever class of work should be actuated by the same high sense of responsibility which prevailed in the present instance, and that both engineers and contractors should co-operate in the execution of honestly built and honestly paid-for work.

WALTER McCULLOH.—A careful watch was kept for cracking in the wall, but up to April 1st, 1893, none whatever had been observed, except the fine irregular cracking generally observed in all pointing.

In my opinion the body of water which was stored behind the dam and in direct contact with it during the severest part of the winters, and which remained there till the latter part of the summers, had the effect of maintaining an almost uniform temperature in the whole mass of masonry and thereby reduced the tendency to crack to a minimum.

Observations were also made, to see if any settlement occurred in the masonry from one season to another; but so far as could be determined there was none.

When new work is to be laid upon old that has set for six months or more, in constructing water-tight masonry, I should not only sweep and clean with water, but should remove a part of the surface with chisel-pointed picks till it has the appearance of rough-pointed work. This was the practice at Sodom Dam and proved entirely satisfactory.

To ensure a uniform mixture of mortars a watcher was stationed over the dry mixing, whose duty it was to see that the proper proportions were put in the box, and that they were uniformly mixed before allowing the box to be taken to the wall. The required quantity of sand was first spread in the box, the cement then spread over it, and beginning at one side the whole was turned three times with shovels. The dumping of the box on the mortar bed effected the fourth turning which the mixture had before any water was applied to it.

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596.

(Vol. XXVIII—May, 1893.

TUBERCULATION IN A 48-IN. WATER MAIN.—DISCUSSION ON PAPER No. 575.

By D. M. Greene, M. Am. Soc. C. E.

The observed facts presented in Mr. Duane's paper are both interesting and important. It is to be regretted, however, that the delivery, both of the Aqueduct and of the several mains, was not more accurately stated.

While fully in sympathy with the author of the paper in his view as to the desirability of simple and reliable formulas over those whose chief merit seems to consist in their complex character, which are generally employed with greatest confidence by those who are largely, or entirely, lacking in such experience as alone can fit an engineer to judge, with any degree of accuracy, as to the proper values to be assigned to certain modifying constants, I long ago became convinced of the unreliable character of the Chezy formula—

$$V = C\sqrt{RI}$$

especially when applied to determine the flow in large channels or

pipes. I assume it to be a well-known fact that the value of "*C*" in this formula varies, between wide limits, with the size of the channel or pipe, and with the velocity of the flow.

Because of this variation I have not relied upon the formula, except where rough approximations to the truth only were required. Where greater accuracy has been necessary or desirable, I have employed a formula of my own construction, which, while simple and readily applied, gives entirely trustworthy results. This formula—

$$V = C \sqrt[m]{R I}$$
$$= C' \sqrt[m]{d h}$$

See errata p. 357.

in which R and I are as used in the Chezy formula, and d and h represent the diameter of the pipe and the frictional head per mile, respectively.

These formulas, or their equivalents, have been used by me for many years, and they will be used in discussing the paper under consideration.

In order to establish the credibility of these formulas, I present a table constructed from observed values, in experiments upon the flow of water in pipes, by Dr. Lampe, in connection with the Danzig Water Works, and by Mr. F. P. Stearns, M. Am. Soc. C. E., in connection with the Sudbury Conduit of Boston water works. These results, together with other data relating to the experiments, were given by Mr. Stearns in a paper read by him, and published in the *Transactions* of the Society, in January, 1885.

The velocities given in Mr. Stearns' tables appear to have been deduced from accurately measured quantities, and are regarded as exceptionally reliable.

In the accompanying table I have inserted "calculated" velocities, for comparison, which have been obtained by the use of the formulas to which reference has been made. I have also inserted values of h, deduced from the values of I, and values of C, deduced from measured deliveries and sizes of pipes.

It will be observed that one of the mains experimented upon had a diameter of 1.373 ft., while the other had a diameter of 4 ft.; also, that the "calculated" velocities ranged from 1.29% above, to 2.65% below, the actual velocities; with a mean difference, or error of —0.805%; also, that the values of C range between 110.7 and 142.11 for clean pipes, both of which, it is understood, were coated.

In making the comparisons, the results in Mr. Stearns' experiment No. 1, which were by him pronounced "anomalous," have been rejected. The results in this experiment are obviously affected by some undetected error, either in the experiment itself or in the records which were kept at the time it was made.

Assuming, now, that there is sufficient warrant for employing the formula which I have constructed, let us proceed to an examination of the questions suggested by Mr. Duane's interesting paper.

First.—What was the probable delivery of the Aqueduct proper, when the area of the section of the flowing stream was 49.24 sq. ft.; R, 2.38 ft., and I, 0.00021?

The author of the paper, presumably using $C=135$, gives us 96 000 000 galls. per day, equivalent to 148.54 cu. ft. per second, and indicating a mean velocity of 3.016 ft. per second.

At the outset we are confronted with the necessity of making a "scientific," or practical "guess"; for our formula, after all, contains a coefficient adapted to the clean-coated surfaces of iron pipes, which must be judiciously modified, in order to adapt it to other surfaces. There is authority for the assumption that our coefficient should be diminished, say, 10%, in order to adapt the formula to such a surface as that of the old Croton Aqueduct is supposed to have; that is, the surface of a plaster of rich hydraulic cement mortar, composed, say, of 2 parts cement and 1 part sand.

Our formula, with its coefficient thus modified, gives for the mean velocity of the flow in the Aqueduct, 3.036 ft. per second, equivalent to a daily delivery of 96 607 400 galls., which exceeds Mr. Duane's estimate by 0.63 per cent.

Second.—What would have been the capacity of the 5 992 ft. of 48-in. main, in Tenth Avenue, experimented upon in 1881, with the loss of head then observed, 3.39 ft., had its inner surface been clean and coated in the usual manner; and what was the efficiency of the actual tuberculated main?

Mr. Duane estimates the actual delivery, by each of the five 48-in. mains then in use, after deducting an estimated diversion from the Aqueduct proper of 3 500 000 galls. daily, at 18 500 000 galls. per day; with an observed loss of head of 3.39 ft. in a length of 5 992 ft., equivalent to 2.987 ft. per mile of main. From this estimate he deduces $C=96$, which is 29% less than "that assigned to it in modern

practice," indicating an efficiency of 0.71, as compared with the delivery of a clean-coated main.

Our formula gives for such a main 48 ins. diameter, and with a loss of head of 3.39 ft. in a length of 5 992 ft., a mean velocity of 3.205 ft. per second, and a daily delivery of 26 026 200 galls. Taking our estimated daily delivery of the Aqueduct proper of 96 607 400 galls., and deducting Mr. Duane's estimated diversion of 3 500 000 galls., we get for the probable delivery of the five tuberculated pipes in 1881 93 107 400 galls., or a mean daily delivery for each main of 18 621 480 galls.

This result compared with our estimated capacity of a clean pipe, other things being the same, shows an efficiency of 0.715 for the tuberculated main, or a loss, in capacity for delivery, of 28.5%, due to fouling of its inner surface.

Let it be noted here that this result, which is practically identical with that obtained by Mr. Duane, is found by a comparison of estimated deliveries in the Aqueduct proper, and in a clean-coated 48-in. main; while Mr. Duane's result is found by a comparison of a value of $C = 135$, for the Aqueduct proper, which has been found to give "satisfactory average results," with a deduced value of $C = 96$; the latter coefficient being 28.9% less than the former. We have used, in estimating the deliveries of the Aqueduct and main, coefficients differing by only 10%, in order to provide for the difference in the characters of the surfaces of the two conduits.

By the use of our formula, we get for the diameter of a clean-coated main, which, with a loss of head of 3.39 ft. in a distance of 5 992 ft., would deliver 18 621 480 galls. a day, or 28.81 cu. ft. per second, 3.516 ft. Thus, it appears that in consequence of the tuberculation, the effective diameters of the mains in Tenth Avenue have been reduced from 48 ins. to about 42.2 ins.; representing a reduction of 22.72% in their effective transverse sectional areas. It appears, also, that this reduction in effective area, effecting a reduction of 28.5% in the capacity of the mains to deliver water, was caused by tubercles none of which exceeded 1 in. in height, and whose average height was probably not much, if any, greater than ½ in.

It follows, then, that of the entire reduction in the capacity of the fouled mains to deliver water, less than one-fifth was due to the reduction in their sectional areas; while more than four-fifths was due to

the increased friction between the water and their fouled inner surfaces.

Third.—What was the probable delivery of the 4 123 ft. of clean-coated 48-in. main in Tenth Avenue and Eighty-fifth Street, with a loss of head of 0.74 ft., and what was the probable efficiency of the 5 306 ft. of tuberculated 48-in. main, with a loss of head of 1.86 ft.?

Here we have for the first time complete data for reliable estimates. Our formula gives, in the case of the clean-coated main, a mean velocity of 1.697 ft. per second, and a daily delivery of 13 750 000 galls., results which are about 5.1% less than were obtained by Mr. Duane, using in connection with the tuberculated main his deduced value of 96 for C in making the estimate.

The values of I, for the sections of clean and tuberculated main, were 0.00018 and 0.00035, respectively; or the loss of head per foot of tuberculated pipe, due to the volume of the water delivered to, and carried by, the clean-coated pipe, with a loss of 0.00018 per foot was 0.00035 ft.

The efficiency of the tuberculated section of the main, as compared with that of the clean-coated section, taken as unity expressed by the mth root of the ratio of the losses of head, we find to be 0.691, indicating a loss in efficiency, due to fouling, of 30.9%, as against 28.5% loss in the case of the other five 48-in. mains which were embraced in the first experiment, and which extended from One Hundred and Thirteenth Street, along Tenth Avenue and Ninety-third Street, to the gate-house at Ninety-third Street and Ninth Avenue, a distance of 5 992 ft.

We find that a clean-coated pipe, 3.471 ft. in diameter, with a loss of head of 1.86 ft. in a length of 5 306 ft., would carry the 13 750 000 galls. estimated to have been delivered by the clean-coated pipe, in the second experiment, with a loss of head of 0.74 ft. in a length of 4 123 ft.

Here we have our effective diameter reduced from 48 ins. to 41.65 ins., by the fouling of its inner surface; representing a reduction of 24.71% in the effective cross-sectional area of the pipe.

The results which we have obtained, supplemented by Mr. Duane's observations or experiment, during the fall of 1892, lead us to the following conclusions:

First.—The maximum effect of tuberculation was produced at the end of seven years, or in 1881.

(*Table to be inserted at page* 357 *of the* Transactions *for May*, 1893.)

Table Showing the Velocities of Water in Pipes, as Determined Experimentally by Dr. Lampe and F. B. Stearns, and as calculated, using the formula $V = C' \sqrt[m]{d^n h}$; with Comparisons of Results.

No.	Authority.	Date of Experiment.	Length of Pipe.	Diameter of Pipe.	Loss of Head due to Friction, in Feet.		Value of C in $V = C\sqrt{RI}$.	Mean Velocity, f. s.		Differences.	
					Per Foot. I.	Per Mile. h.		Deduced from Measured Flow, $V = \frac{Q}{A}$.	Calculated by Formula, $V = C' \sqrt[m]{d^n h}$	Absolute.	Percent.
1...	Dr. Lampe	October 1, 1870		1.373 ft.	0.0005915	3.123	110.7	1.577	1.564	— 0.013	— 0.82
2...	"	" 8, 1870	4½ to 6 miles.	1.373 "	0.001376	7.265	114.1	2.479	2.498	0.019	0.77
5...	"	March 19, 1871		1.373 "	0.00163	8.606	114.6	2.709	2.744	0.035	1.29
4...	"	October 17, 1869		1.373 "	0.00195	10.296	119.4	3.090	3.031	— 0.059	— 1.91
5..	F. B. Stearns	May 1, 1880	1 747.2 ft.	4.0 "	0.0003181	1.68	146.67	2.616*	2.327	— 0.289	— 11.05*
6...	"	" 5, 1880	1 747.2 "	4.0 "	0.0007115	3.76	140.14	3.738	3.639	— 0.099	— 2.65
7...	"	April 26, 1880	1 747 2 "	4.0 "	0.0012206	6.44	142.11	4.965	4.905	— 0.060	— 1.24
8...	"	" 27, 1880	1 747 2 "	4.0 "	0.0018485	9.76	144.09	6.195	6.178	— 0.017	— 0.27
						Means, *including* "anomalous" result, No. 5... ...				0 067	— 1 985
						" *excluding* " " No. 5.......				0.035	— 0.69

* Mr. Stearns designated the result in the 5th experiment recorded in this table as "anomalous." We have *in*cluded it, and then have *ex*cluded it, in determining means.—G.

ERRATA.—Vol. XXVIII.

The formula on page 353 should read—

$$V = C \sqrt[m]{(4\,R)^n\, I}$$
$$= C' \sqrt[m]{d^n\, h}$$

Second.—This maximum effect appears to have been a reduction of about 30% in the capacity of the 48-in. main to deliver water.

This conclusion is identical with one of the conclusions reached by the author of the paper, by a different and, as we think, less satisfactory process.

Thrid.—Coated mains, 42 ins. diameter, would have served as well as the 48-in. uncoated pipes, and would have saved to the City of New York the difference in the cost of the two sizes of pipe, together with interest upon that difference, for an indefinite period.

Fourth.—That it is not safe to rely, in important hydraulic works, upon assumed or deduced values of *C*, in the Chezy formula, since its value is affected by three ever-varying conditions, namely, size of pipe, velocity of flow in it, and the condition of its surface.

In conclusion, I beg to offer to Mr. Duane my personal thanks for the interesting and valuable facts which he has observed and presented in his paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
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(Vol. XXVIII.—May, 1893.)

RESTORATION OF THE CABLE ENDS OF THE COVINGTON AND CINCINNATI SUSPENSION BRIDGE.—DISCUSSION ON PAPER No. 578.

By F. Collingwood, L. L. Buck, H. B. Seaman, John Thomson, A. J. Frith, S. Whinery, H. S. Haines, H. F. Dunham, E. B. Gosling, E. P. North, Theodore Cooper and G. Bouscaren.

F. Collingwood, M. Am. Soc. C. E.—The paper by Mr. Bouscaren is a very interesting one, and describes clearly the novel method pursued in repairing the damage to the cables which had been caused by rust. Such experiences emphasize the importance of so constructing the vital parts of any bridge that they can be examined at all times without difficulty.

There is one point in the paper upon which further information is desirable. I refer to the statement respecting the friction available from the clamps on the cables. This seems to have been determined upon the supposition of a free movement of the wires upon each other, and that the pressure effective in producing friction will have the same ratio to the sum of the strains in the bolts as the circumference of the cable has to its diameter. It must be conceded that there will be

a total pressure of 540 tons each side of the diametrical plane at right angles to the bolts. Hence, with a coefficient of friction of 0.25 there may result: $540 \times 2 \times 0.25 = 270$ tons.

This is considerably less than the amount taken; it involves also the question as to just what value the coefficient of friction has. I would ask whether experiments have been made in this direction. The experience of engineers in using clamps upon wire rope is that it is a difficult matter to obtain a firm hold in that way.

In 1883 I was called upon to examine and repair the cables of the Allegheny Suspension Bridge. The cables entered the masonry but about 3 ft. below the surface of the sidewalks, and were subject, therefore, to considerable alternations of temperature. The ends were enclosed in canvas, the spaces being then filled with coal tar, and the whole surrounded by concrete and masonry. The heat in summer caused the coal tar to ooze away in part into the masonry, and then, by slow condensation, the space became filled with water which contained considerable sulphur and carbonic acid, resulting, doubtless, from the smoke of the region. Many of the wires were deeply pitted and others rusted off. Fortunately no great damage had been done to the wires at and near the shoes, or the method of repair by splicing new pieces into the damaged wires would not have been adopted. The splices were made by filing sloping faces 3 ins. long on the ends to be spliced, then nicking the convex surfaces, lapping the ends and wrapping them tightly for 4 ins. with finer wire.

A special apparatus was devised by which an extra tension was given in the wire outside the apparatus, so that, when the splice was completed, the proper tension remained in the wire. The tension was in every case tested by pulling the wire from a straight line a measured distance, and noting with a small spring balance the power required to do this. After a little experience the workmen were able to so adjust the lengths that they rarely missed obtaining the proper tension at the first trial. Several special tools were devised during the progress of the work, and full details will be found in the paper mentioned in the foot-note.*

To prevent damage in the future, tunnels with manholes were built around the cable ends, and the latter beyond the wrapping and within

* See paper "On Repairing the Cables of the Allegheny Suspension Bridge at Pittsburgh," by Francis Collingwood, M. Inst. C. E. Minutes of Proceedings of the Institution of Civil Engineers, Vol. LXXVI, p. 334.

the masonry were entirely enclosed in best-quality paraffine wax. This was poured into a temporary mold while very hot, so as to surround every wire closely.

A piece of the old wire which was free from pits or rust was tested, and gave substantially the same stretch and strength as new wire; and there was no evidence that any of the wires had been changed or damaged other than by rust.

The diameter of the tested wire was 0.144 in., giving an area of 0.1626 sq. in., and its breaking strength was 1 450 lbs., or at the rate of 89 015 lbs. per square inch. Under a stress per wire of 200 lbs., increased by successive additions of 100 lbs. up to 800 lbs., the stretch increased 0.000210 to 0.000275 ft. for each 100 lbs. added. The stretch at rupture was 0.0208 ft. or ¼ in. in 1 ft. A new Bessemer steel wire of about the same diameter gave practically the same stretch, but it was not so regular. The breaking strength was 90 422 lbs. per square inch. The elongations were all measured by Paine's multiplying gauge.

L. L. Buck, M. Am. Soc. C. E.—Mr. Bouscaren's description of the condition of the ends of the cable strands of the Cincinnati Bridge would have applied equally well to that of the Niagara cables' ends, at the time they were inspected in 1887. The same want of care manifested in permitting an occasional spawl of limestone to come into contact with the wires, which ate quite through some of them and the crust of rusted mortar, is called to mind very vividly.

The appearance suggested, to my mind, that of a partially uncovered Egyptian mummy, and the feeling experienced in starting to remove it was not much different from that of an archæologist in uncovering a valuable specimen of those very cheerful objects.

I found that the removal of the crust was greatly facilitated by thoroughly soaking it with kerosene oil, frequently applied, for a day or two, and then chipping it away with a chisel. In this way, quite large chunks would come off. Great care was used in chipping; and by always keeping the edge parallel with the wires, it was accomplished without injuring any of them in the least.

In the case of the Niagara cables, the seven strand ends were each looped around a cast shoe. Four of these shoes alternated with five eye-bars, and were connected by a 3½-in. pin. The other three shoes were connected in the same manner with four eye-bars by a 3½-in. pin and placed above the first set.

The mortar used in covering the strands was composed of sand and Thorold cement, and was very good. It is my opinion that Mr. Roebling's idea of the protective effect of cement mortar upon metals imbedded in it, under proper conditions, was in the main, correct. In cases where no spawls came in contact with the iron, and there was no movement of the metal in the mortar, it was entirely bright after 25 years. To be sure, the iron had been painted before covering with mortar, so that it is possible that the mortar only protected the paint and that in turn the iron.

Aside from the spawls, the difficulty in the Niagara case arose from the fact that the strands elongated considerably with every load that went upon the bridge, and the rigid masonry not being able to go with them, there was a sort of porous portion immediately surrounding the group of strands, and this admitted water which the cables had conducted to it. The portion of mortar between the strands had conformed more to their movements. Consequently, the main injury was confined to the outer sides of the outer strands of the groups.

Beginning at the band, where the divergence begins, the etching was slight, but went quite around the cable. Thence toward the shoes it appeared to increase on the lower strands and become less on the upper, till at the shoes it was the worst on the under side where the loop of the lower strands began.

This portion must have been damp most of the time.

Tests of specimens cut from defective wires showed very similar results to those given by Mr. Bouscaren. The stretch was considerably less than that given by Mr. Roebling in his report of original tests of the same, but this is easily accounted for by the fact that the etchings served as nicks to concentrate the stretch to themselves, and mainly to the weakest of such points. There were no indications discoverable that the metal had undergone any molecular change.

I was much interested in Mr. Bouscaren's description of his method of re-enforcing the strands. The method of measuring the stress on the new bars was certainly very ingenious, and is doubtless practically accurate.

Mr. Bouscaren does not state whether he examined the pins in the shoes, to ascertain if they were bent. In the Niagara instance they had a general bend from end to end and convex toward the towers. This

was due in part to the outer bars of the chain having, if anything, slightly greater section than the intermediate ones, while they had very much less stress, and to the fact that as far as the succeeding joints of the chains were uncovered, they were found badly coupled. The result was that the intermediate bars had elongated considerably more than the outside ones.

It is necessary to note two essential points of difference in the two bridges.

First.—In the Cincinnati Bridge, the cables support half spans between the anchorage and towers, while in the Niagara, this portion sustains no vertical load excepting its own weight, and hence passes more directly from the anchorage to the tower. In both cases the stress is greater at the tower than at the anchorage. But this difference is much greater in the case of the Cincinnati than in that of the Niagara. The difference in any case is accurately measured by the difference in the secants of the angles made by lines tangent at the two points with the horizontal.

Second.—The Cincinnati, being for common traffic, will seldom receive its maximum loading, and that will always move upon it very gradually, while the Niagara receives its maximum allowable load many times each day, and it frequently moves on in half a minute.

Now, if the wires in the cable are so firmly gripped by the band at the divergent point of the strands and by the wrapping that they cannot slip, and thus weaken the cable at the tower, a certain number could be cut at the anchorage and not affect the sustaining power of the cable. At Niagara, the difference between the stresses at towers and anchorage was not very great, and there was also reason to doubt whether the severed wires might not have slipped, so as to lose a portion of their stress at the towers. If so, there was no other way to do except to splice each wire and give it its individual share of the stress. This was successfully accomplished as described in my paper on that work, and would take too much time to repeat here.

In the case of the Cincinnati Bridge, a greater portion of strength at the ends of the cables could have been dispensed with, but not having the necessary data, I have not estimated it. With the main band at the point of divergence of the strands, and the numerous cable bands, all gripping the cable tightly, it is not probable that any of the wires slipped, so as to relieve them of their share of the stress at the

towers. The numerous clamps put on by Mr. Bouscaren will effectually prevent slipping hereafter. Whatever the loss of strength at the diverging strands exceeds the difference of stress between tower and anchorage, it will be satisfied, and if the bars have received a stress in excess of that, they will be a benefit in case the outer bars of the chains are as large as the intermediate ones, as the new rods will transfer a portion of the stress from the intermediate to the outer ones.

Mr. Collingwood.—There is one bit of experience I had on the Allegheny Bridge which may be of interest in other cases. The bridge had been painted so much that there was a thick coating on the cables which was full of circumferential cracks. To paint over these with a thin coat of paint was almost sure to lead to a crack again in the same place. I reported to the proprietors of the bridge that it was desirable to remove the old paint before repainting, and received orders to do it. I found I had a contract on my hands. The first thought was to scrape it, but that did not work, on account of the gumminess of the covering. The painter thought he could burn it off, but soon found this utterly impracticable. The thought then occurred to me that it could be sliced off, and a broad chisel was procured for the purpose. The first cut with this took more off than we had been able to do with five minutes' work the other way. A foot in length of the cable was cleaned, so as to get some sort of a standard of the time required, and each man was required to clean 15 ft. per day. As it was rather difficult to cut entirely around the cable with a chisel, drawing knives were obtained; so with chisel and drawing knives and a grindstone, keeping one man sharpening all the time, we took the paint off. We had no trouble after we discovered how to do it, and it was a great improvement on any other method.

H. B. Seaman, M. Am. Soc. C. E.—Did you ever try ammonia?

Mr. Collingwood.—It would have been a big job to clean 2 000 ft. of cables in that way.

John Thomson, M. Am. Soc. C. E.—How thick was the paint?

Mr. Collingwood.—It was nearly $\frac{1}{16}$ in., as I now remember.

Mr. Buck.—The Niagara cables were the same way, only not to that thickness. They were cracked a good deal when they were using white paint. I found that zinc paint removes that difficulty better, keeping its flexibility longer. We have used zinc paint for some 10 or 12 years now.

A. J. Frith, M. Am. Soc. C. E.—I would like to ask Mr. Collingwood about the difficulty of holding the clamps on the wire cable; what methods were used in fastening those clamps and what were the results?

Mr. Collingwood.—To what clamps do you refer?

Mr. Frith.—You were speaking, in your discussion that you read, of the friction of the clamps, etc., the difficulty of holding the clamps.

Mr. Collingwood.—It will be remembered that the ropes for supporting the cradles and the footbridge were from 2¼ to 2¾ ins. in diameter. In hauling them up to the anchorage, after passing them over the towers (while the tension was light), they used clamps; inside these were cast so as to fit the twists in the rope, and they were fastened together with screw-bolts and nuts. My notes show at least two cases of slipping of these clamps on the ropes; the hold which never slipped was that of the chain or chains fastened by hitching, to use the sailors' phrase. One of the heaviest ropes, I remember, had the chain fastened at the draught end by, first a rolling hitch, then three rolls, followed by a half-hitch, and the free end finally seized to the rope. A plan sometimes pursued is to pass a rope around a curved eye, and then to clamp the free end to the standing part of the rope by clamps fastened together with bolts and nuts. Undoubtedly, the most secure fastening is that of the conical socket, secured on the end of the rope by means of long, tapering pins driven in between the wires.

Mr. Buck.—It is much more difficult to take a large bundle of wires and anchor them by means of a clamp. That is the case with a wire rope. The inner wires of the bundle will, many of them, receive little pressure from the clamp, and, as they are all parallel, it is not practicable to hold them all securely. In the case of the rope, the twisting has somewhat the effect of interlocking; but even a rope of any considerable size is not easily anchored by a clamp.

Mr. Frith.—It is a question of pressure.

Mr. Buck.—You may get enough pressure on the outer wires to hold them, but the inner wire will not be held by the clamps.

Mr. Frith.—Is it not a question of the amount of pressure?

Mr. Buck.—Yes; if you get pressure enough on a wire it will hold it.

Mr. Frith.—The idea is that the requisite pressure cannot be obtained with the clamps ?

Mr. Buck.—Yes ; it will not properly secure the rope. In Mr. Bouscaren's work probably most of the broken wires were around the outside of the bundle, and in that case the numerous clamps gripping them directly would probably secure them. But if they had slipped before the clamps were put on, the latter could not restore the stress such wires had lost by slipping.

Samuel Whinery, M. Am. Soc. C. E.—I had the pleasure of talking with Mr. Bouscaren while he was engaged on this work, but did not visit it when in progress. I am able to answer one of the questions that has been asked. That is, in reference to the pins. The pins are larger than in the Niagara Bridge. What was the diameter of the pins in the Niagara Bridge ?

Mr. Buck.—Three inches.

Mr. Whinery.—I think Mr. Bouscaren told me that the pins were apparently uninjured and apparently not bent in any way. Mr. Bouscaren, when the work was in progress, some months since, gave a valuable account of it at a meeting of the Cincinnati Engineers' Club, and the subject received some discussion at the time. The question of the extent to which the clamps took hold of the cable was discussed there. The point was raised, as a possible fact affecting the friction, that as a matter of fact the rings or clamps did not take hold around their whole surface, since, as the wires were cylindrical, they only touched each wire at a small portion of its surface. It used to be the theory that friction was independent of the area of the surfaces in contact, but recent experiments show that it is not strictly true. To what extent this fact would affect the holding power of the clamps I am not prepared to say. There may be a possibility of more or less motion in the wires, due to expansion and contraction under tension and difference of load, and as these clamps take hold only of the outer wires of the cable, it must throw a largely increased proportion of the strain on those wires.

There is another matter I should like to ask about. I recollect seeing in one of the technical papers, a few years since, that some chemical, which I think was chloride of zinc, had the property of removing entirely iron rust, without in any way affecting the metallic iron. I don't know whether I am correct about the substance being

chloride of zinc. I would like to ask if any member here has any information on that point?

Mr. Collingwood.—I would like to ask Mr. Whinery whether I understand him rightly, that the clamps used by Mr. Bouscaren were put over the wrapping, and did not come directly in contact with the parallel wires of the cable? How does he know, in that case, exactly what pressure he gets on the interior wires?

Mr. Whinery.—I think I was correct in saying they were so placed. I think the wrapping was not removed.

Mr. Collingwood.—The point I want to get at is, can the exact pressure on the interior wires be known.

Mr. Thomson.—Is it not a question if you get any?

Mr. Buck.—He may get some pressure as the cable bands are bound around the cable tightly, it just brings those wrappings around and makes them bulge where the suspender hitches in; it occurs to me that the wrappings there have some general bearing on the wire; they might compress into each other and make them hold more effectively still. That might make it hold a little stronger.

H. S. Haines, M. Am. Soc. C. E.—I don't know that it has any very direct bearing upon this particular question, but it only occurred to me to mention that we have found cement to be a valuable protection for a ship against oxidation from bilge water. I have seen it where these plates had been for four or five years under the bilge water, when they were just as bright under the cement as when the ship was built. This has been the experience in the American and British navies, that cement applied to the surface of plate in the bilge water under the engine-room and fireroom does protect them against oxidation. Of course, in this instance, if the movement of the cable wires caused them to break away from the cement and permit water to percolate into them, its value as a protection would be lost.

Mr. Whinery.—Mr. President, I have imagined that the effect of mortar upon iron depended somewhat upon the constituents of the cement. Cements that contain a very small amount of, or no, sulphur, would, I imagine, protect the iron pretty thoroughly. But I think a small percentage of sulphur, which all our natural cements contain (and I believe natural cement was used in both the Niagara Bridge and the Cincinnati Bridge) might account for the corrosive action of the cement to a great extent.

The CHAIR.—What would be the effect of the lime in the cement?

Mr. WHINERY.—I do not think it would have a deleterious effect.

Mr. BUCK.—The cement that Mr. Roebling used was Thorold cement. On the tops of the towers, the saddles and the cables lying in them were covered to a considerable depth with cement mortar and when it was removed the wires were perfectly clean and bright. They are so still.

Unfortunately the mortar had worked down among the rollers under the saddles and formed little troughs in which the rollers laid, holding the water which leaked in. The result was that the under sides of the rollers were flattened, and the bed plates grooved by rust. This prevented the rollers from moving; consequently the towers had to bend to satisfy every movement of the cables. The towers became so badly cracked that they were replaced by iron ones.

H. F. DUNHAM, M. Am. Soc. C. E.—I am surprised at these facts about the masonry, as I had supposed that only the best of material and workmanship could have been introduced where the integrity of the work was so important.

It would be of interest to know whether like imperfect work extended to the anchor chains and pins at the different places, and, if so, what effect it had there produced ?

The CHAIR.—Can Mr. Buck give us some information ?

Mr. BUCK.—All I can say is that we uncovered about four lengths of the anchor chains in the Niagara Bridge, and the pins and bars there were entirely free from rust. In one place the bars and chains had been painted, and in picking off the cement that had covered them quite a chunk would come off, and underneath they looked as bright as new. The bright end of a pin that had been filed 25 years before looked just as if it had been filed that day. How it might be down below, where there was sulphur water, etc., is a little uncertain. The condition of the pins, chains, etc., decided us to put new anchor chains in and renew the whole casing, which was done, 50 sq. ins. at each end of each cable. The question of the bending of the pins was one thing that caused a good deal of inquiry; we thought of it a good deal; that is, as to what was the cause of it, and how far down it extended. Whether it was an indication that bad work was going on below was a question, and all those things decided us to put in the new anchorage. The whole thing was done by sinking four anchor pits 23

ft. deep on the Canadian side, 17 ft. deep on the American side, and 6 by 2½ ft. in plan. The new anchorage complete cost $26 000.

Edgar B. Gosling, Jun. Am. Soc. C. E.—I noticed something peculiar on some work of mine in Egypt about iron imbedded in béton. I found that if the béton was not remoistened until sometime after it had set, the iron imbedded therein did not rust. If, however, the béton was moistened again before it had set sufficiently, the iron rods did rust. I mention this, as it might be of some interest in connection with the discussion of the way iron acts when used with cement.

Mr. Collingwood.—I would add, in reference to what Mr. Dunham says, that the larger the anchor bars, the more trouble there is from black oxide remaining on them as they come from the shops. I believe it is a fact that this is liable to scale off afterwards, so as to remove any protection that may be put upon the surface. In the East River Bridge work, where the anchor bars were very large, the scale on the heads was over $\frac{1}{32}$ in. thick, and was too heavy to be removed by acid without eating the bars; the bars were, therefore, first put in hot potash water heated by steam for about one-half hour, to remove any grease upon them; they were then taken out, and the thick scale hammered off. They were next treated by sulphuric acid for four hours, then dipped in hot lime water and rinsed and brushed. While they were still hot, raw linseed oil was applied; this was followed by raw oil and Spanish brown, and then by a coat of boiled oil and Spanish brown, the whole operation occupying about a week. When placed in the masonry all spaces were filled with rich, pure cement grout made of Rosendale cement.

Mr. Buck.—I know if a limestone spawl comes in contact with iron, that part is going to rust. At Niagara the depth of $\frac{1}{8}$ in. was in such cases eaten away. This is always indicated by black spots on the surface; where you see one of those black spots you may know the spawl there has been in contact with the iron.

E. P. North, M. Am. Soc. C. E.—Is limestone worse than anything else?

Mr. Buck.—Worse than anything that we found at Niagara.

Theodore Cooper, M. Am. Soc. C. E.—As far as this case goes, the faith of Mr. Roebling and other engineers in the preservative effects of cement mortar need not be shaken, for, from the author's description of the masonry and the wooden blocks and chips imbedded

therein, the cables were not imbedded in cement mortar. The examination of these cable ends, and those heretofore made on the anchorages of the Niagara and Allegheny bridges, all tell the same story—corrosion and loss of strength. That cement mortar has in some cases proved a protection against oxidation cannot always give us perfect assurance that, in any particular case, we have secured the exact conditions necessary to prevent corrosion. These conditions, as we understand them, are total exclusion of acid substance, or of any material which may by its own changes produce any acid action. While a total exclusion of air, water and acids, or materials that may produce acids, will render oxidation impossible, the obtaining of such conditions in general practice is one of great difficulty. That the best and most compact masonry will allow the passage of water, sufficient at least to produce a moist condition is undoubted; pure water, however, does not cause oxidation, the presence of an acid is considered as also necessary. That water percolating through cement has any acids contained therein neutralized is the natural explanation of the preservative character of cement mortar. If, however, the cement covering be imperfect, so that any water can pass without this neutralizing of the acids, oxidation must be expected.

The method selected by the author for the treatment of this particular case, appears the best possible solution, provided the paraffine is pure and can be maintained in its position.

The attachment of the additional auxiliary bars to the cable by friction clamps was perfectly proper in this case, being apparently the only practical method; but it brings up a question which has a bearing upon all cable practice, viz., what is the effect upon the resisting power of a cable from the pressures due to these frictional attachments?

As the present system of suspension bridges requires the use of such attachments, it would be well to determine by actual experiments if with such attachments the strength of the cables is unaffected.

G. Bouscaren.—Referring to Mr. Collingwood's remarks: I do not know of any experiment having been made to determine the ratio between the pressure and friction in clamps on cables of suspension bridges. The coefficient of friction between smooth surfaces of iron is given as 0.19 by Morin; between the rough surfaces of the wires

and clamps, it should manifestly be much larger, the coating of sand between the two surfaces should give a better grip than obtainable between the driving wheel of a locomotive and rail, where the coefficient is generally admitted to be not less than 0.25.

The supposition that the friction may be reduced to 260 tons by assuming the pressure to be applied only in a direction parallel to the clamp bolts is scarcely admissible from the fact that one effect of the tension of the clamp bolts is to close the two holes of the ring by bending in a direction at right angles with the bolts, the clamping action on the cable taking place in two directions at right angles to each other.

The wrapping of the cables was not removed for two reasons: 1st, because it was not considered advisable to deprive the wires of the protection afforded by the wrapping; 2d, because it was thought that the interposition of the wrapping between the clamps and the wires would increase the frictional resistance.

The method of attachment by clamping is not entirely satisfactory in respect to the uncertainty which exists as to the participation of the internal wires of the cable to the stress transmitted by the clamps, but the choice was limited between such an attachment and the splicings of the injured wires. The latter plan was considered under the circumstances as impracticable and still more uncertain in its result.

The broken wires were carefully observed while the strands were being cleansed and before the clamps were placed in position, to detect any slip which might occur ; no longitudinal motion whatever could be detected, and it is scarcely admissible that any could take place after the clamps were put on.

Referring to Mr. Buck's question as to the condition of the pins; it was impracticable to determine whether they were bent or not, as the ends only were accessible to view, the cast-iron shoes and the heads of anchor bars forming a continuous casing over them.

Owing to the porosity of the masonry and the fact that it was generally found saturated with moisture, no conclusion can be drawn from the experience in this case as regards the preservative action of cement mortar on iron, when air and water are excluded. In 1876, when the bridge over the Kentucky River was built for the crossing of the Cincinnati Southern Railway, several links of the anchorage of

the Suspension Bridge which was partly built by Roebling at the same place in 1855 were dug out and were found in a perfect state of preservation, not a spot of rust being apparent on the bars; but the mortar in which they were imbedded was very compact and dry and of excellent quality.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

598.
(Vol. XXVIII.—May, 1893.)

JETTY HARBORS ON THE PACIFIC COAST.—DISCUSSION ON PAPER No. 584.

By GEORGE Y. WISNER, G. H. MENDELL, B. W. DECOURCY, WILLIAM A. JONES, THOMAS W. SYMONS, Gen. D. S. STANLEY, WM. M. BLACK, CHARLES MACDONALD and A. J. FRITH.

GEORGE Y. WISNER, M. Am. Soc. C. E.—The admirable paper with which Captain Symons has favored the members of the Society brings out in a very interesting manner the advances that have been made in method of jetty construction on the Pacific Coast, and the achievement of results that reflect great credit on the officers who have had charge of the works. They are to be congratulated for having discarded the custom prevalent in the East, of doing everything by contract, and for having executed the work themselves whenever success and economy have required it. The common practice of readvertising work when bids exceed the engineer's estimate—even when, as is often the case, that the engineer could not complete the work for his own figures—is demoralizing, and generally leads to bankruptcy on the part of the contractor, or the acceptance of work that should be a disgrace to the profession.

It is to be regretted that Captain Symons has not given the amount and cost of material a little more explicitly, as such matters are often

of great value to engineers for the purpose of correctly estimating the probable cost of other works of a similar nature. For instance, at Yaquina Bay, the average cost per linear foot of jetty is given as $90; yet from an examination of the chart it would appear that 30 tons per linear foot would probably be more than the average amount of material used in the construction of the jetty. The cost of stone in the jetty for the past year is given as 82 cents per ton, and assuming the average price for the entire work to have been $1, we should have the average cost of material in place for 1 lin. ft. of jetty, as follows:

Thirty tons of stone	$30 00
Tramway	6 00
Office expenses	4 00
Total	$40 00

It would be interesting to know why the unknown expenses should exceed the cost of material in place by 25% of the latter.

The original estimate for this work was $465 000, or $170 000 less than the amount thus far expended.

At Columbia River the average cost per linear foot of jetty is given as $83, or about double the cost of the material in place, as indicated by the amounts and prices given.

As stated by Captain Symons, this work has been completed at about one-half of the original estimate, but this is accounted for by the fact that in the original estimate rock was figured at $5 per cubic yard, or nearly five times the price actually paid, and also that the outer 7 500 ft. of jetty was to be built of concrete blocks of 10 to 40 tons weight each.

In regard to the plan of the work at Yaquina, there is some question whether a better result might not have been obtained by making the outer portion of the jetties parallel, instead of converging. In fact, the chart rather indicates that an extension of 2 000 ft. may yet be needed before the channel across the bar is what it should be, and in such event the extended jetties should certainly be parallel. The advantage gained by such construction is that the volume of discharge acquires momentum in direction of axis of channel, and is consequently effective in producing scour for a considerable distance beyond the jetty ends.

As to the necessity of the jetty channel being of sufficient width to

allow the tidal basin to fill rapidly, there may be a difference of opinion.

When the pass between the sea and the tidal reservoir is large as compared with the latter, the difference in level will always be small; but when the pass is so small that the level of the reservoir remains practically constant, the difference of level becomes a maximum and nearly equal to a half tide. It therefore seems evident that to obtain a maximum scour from tidal effect, the channel must be made as narrow as safe entrance and good navigation will allow. Undermining of the jetties by strong currents may always be prevented by short spur dikes, constructed at such intervals as may be required along the channel side of jetty walls. The spurs also have the effect of building up a sloping bank, such as to materially strengthen the jetty against wave action from the outside.

In case the jetty enrockments are beaten down by wave action, it is proposed to rebuild them with blocks of stone or concrete of sufficient size to resist wave action. Such work requires very heavy and expensive plants, and the question naturally arises, why not construct a concrete wall in place, which can be done more cheaply and at the same time, of blocks of much larger dimensions than could be handled with derricks?

As to the improvement of harbors by means of a single jetty, the writer's experience in that direction has not been of an encouraging nature; and the fact that water flowing from a tidal pass will find the sea level over the shortest line of least resistance will probably cause such attempts to terminate in failure in the majority of cases.

The fact that sand accumulates on the outside of both jetties simply indicates that, if either jetty be removed, the sand will soon find its way into the deeper portion of the channel, especially if disturbed by waves.

The plan of building jetties up gradually from the bottom along a considerable distance is all right, provided the mattress bottom protection be kept close up with the completed trestle-work, otherwise wave action around the piles may increase the depth to such an extent as to require double the amount of material to construct a jetty that would be required if the completed work be kept well to the front.

G. H. Mendell, M. Am. Soc. C. E.—This paper is a clear presentation of the general conditions of the Pacific Coast, and of the

particular conditions attendant upon the instances of harbor improvement therein mentioned.

Success, in some measure or other, has attended all attempts to better the harbor entrances on the Pacific Coast. This is particularly true of the three instances in which improvements are well advanced, namely, Wilmington, Yaquina, and the Columbia River. These ports all open upon sandy coasts. This condition applies to all other points on the west coast where works are in progress, or for which they are contemplated. The sand is mobile, and may be said to be always in motion, by wind, wave or current. In the natural condition of an unimproved entrance, it lies in great deposits, forming bars and spits, bare or with little depth of water, between which one or more channels make out to sea. In these positions, sand is peculiarly exposed to the action of shifting currents, and to displacement by waves, to the injury of adjacent channels.

A noticeable feature of the bettered entrance is, that the deposits of sand have in the course of the improvement been moved in great part to new positions behind the jetties, where it is more or less securely impounded. In these positions it is least liable to displacement, and if the jetties be sufficiently high, neither wave nor current can place it again where it can be injurious to navigation.

The storage function of these works is referred to in this paper as secondary. It is, however, of great importance—not yet capable of full estimation—in withdrawing or lessening the amount of the element, which, when in circulation, is a great obstacle as well as a cause of impairment of the scouring force, but which, when properly stored, is a bulwark, adding strength and safety to the slight works which produce the result, and is also a directrix of the tide.

It follows that the mounds of "*pierres perdues*" ought to be high, always at least to high water, unless there be such exposure as shall flatten out the walls; and where there is no exposure, they ought to afford the maximum storage capacity, and to this end be several feet above high-tide level.

The magnitude of this phenomenon of moving sand is strikingly illustrated in the instance of the Columbia River. The estimate of the cubical contents of sand deposits made to the south of the line of works, during the progress of the improvement, is about 25 000 000 cu. yds.

In comparison with this result, and also with the great improvement of the bar channel, the cost of the Columbia work—less than $2 000 000—is worthy of attention as an illustration of the inexpensiveness of these works. Another important circumstance is brought out in this paper, namely, that the main circulation is in all these instances that of the tide, and that backwater is always unimportant, even in the Columbia River, or insignificant or absent, as in the instances of Yaquina and Wilmington. The form of the phases of the diurnal tide is peculiar, in that generally it requires two periods of flood to reach the high-water stage, and that both of these flood tributes escape in one large ebb, attended with increased velocity and scouring power.

It will also be noticed that the rise of tides on the Oregon coast, at Yaquina and the Columbia, is for the mean about 7 ft.

The tidal conditions are therefore favorable for improvements.

The lightness of construction of these works must attract attention.

The stones of which they are made rarely exceed 5 or 6 tons in weight. They are mostly in smaller sizes. Indeed, graduated sizes make better work.

The works are protected from wave blows by outlying sand deposits. They do not seem to be threatened to any noticeable extent on the weather side.

No one of these works is finished. Experience with them is less than 10 years. They have so far admirably answered demands, and they have been inexpensive. It remains to ascertain what the future has in store for them, and what measures shall be needed to protect and preserve them.

Bolton W. DeCourcy, M. Am. Soc. C. E.—Engineers are to be congratulated that Captain Symons has given the Society a description of some of the interesting work being done by the Corps of Engineers on the Pacific Coast.

His statement of the limited knowledge of the currents is quite correct; the statements of masters of vessels navigating these waters are contradictory, and, in some cases, seem to be incorrect, by the results observed along the shore.

I have been taking some observations of that part of the coast of Washington lying north and south of the mouth of the Chehalis River, for some 20 miles in each direction, and these do not seem to cor-

roborate the assertions of those masters with whom I have conversed on the subject of these ocean currents.

Some of these gentlemen assert that the great Japan current flows south in summer, but to the north in winter.

Last winter (1891–92) the British ship *Ferndale* was wrecked about 10 miles north of the mouth of Gray's Harbor. She was coal laden. In a short time this coal could be picked up for 20 miles south along the shore.

Last summer a large dead whale was drifted south in plain sight of all the people on the beach.

This winter the wreck of a vessel called the *Big River*, during the prevalence of the most violent storm that we have had in the entire season, and that from the southwest, was raised on a high tide and drifted a half mile to the south in the teeth of the storm.

The embouchure of the Chehalis River, where it enters the ocean, seems to be deflected to the south under the influence of the current, and does not vary materially during the seasons of a year. This channel is phenomenal; it never has received any attention from the Government, and has a depth at low tide of 16½ ft., and has had vessels drawing 17 ft. passing out for years.

If this harbor ever gets an appropriation for the improvement of the bar, it would be the very place to try the single jetty, curved, and concave toward the channel, which I have great faith would result as expected by Captain Symons.

Major WILLIAM A. JONES.—It is assumed that the object to be sought in this discussion is the evolution of experience, results and facts such as bear upon the creation and betterment of ocean harbors, and that it should be no love-feast, in which success alone is displayed and lauded. Hydraulic engineering is yet in infancy, and engineers have much to learn and nothing to fear from their shortcomings.

The ocean shore of the North Pacific is the scene of two clashing factors which very much simplify the problem under discussion. They are:

(1) The great wave-power developed by a rapidly falling foreshore which looks out upon an extended stretch of free ocean.

(2) The extraordinary volume of littoral sands.

To the front the ocean stretches away for thousands of miles, and the deep water bluffs hard up against the shore. The great wind-

waves roll in unchecked until close upon the shore. Here they break upon the outer slopes of sand in very great depths. It is authentically reported that on the outside bar at the mouth of the Columbia River they break at times in depths of at least 80 ft. of water. Obviously, this creates waves of translation of extraordinary moving power. Indeed, in the face of such power alone it is doubtful whether we could make structures within reasonable limits of cost to withstand them. But at this point the other factor enters with its forces in our favor. I will lead up to it with a generalization. The shore of the South Atlantic coast is in a transition state. The geologists call it a condition of continental depression. It is slowly sinking. As a consequence, the ocean is steadily creeping in upon a low shore whose foreshore is long and very flat. Here, the great ocean waves are tripped far outside and broken down to a comparatively small size before reaching the site of our harbor constructions. And, although there is considerable littoral sand in movement, it is not being increased in volume to much extent. On the North Pacific coast just the opposite conditions at present obtain. There, a continental elevation is undoubtedly going on, slowly raising the shore. This presents it at such a slope to the water action that the latter is rapidly carving it away, leaving a shore of lofty, precipitous bluffs. A considerable portion of these are composed of soft sandstone, but it makes little difference to the mighty power beating against them; all are crumbled and dumped into the water to become littoral ocean sands. The volume of these is something stupendous. In places they are forced up on the beach at high tide, caught by the winds at low tide, and blown ashore, forming great seas of barren, waving dunes.

It is the pitting of these two factors against each other which has rendered jetty construction a practical success on the North Pacific coast. It is only necessary to place such structures as will trip and throw down the great mass of moving sands, and they may be relied upon to quickly lay themselves alongside and defend the structure against the waves by means of a long, gently sloping foreshore of sand bars. Expensive structures have not so far appeared necessary. A structured mass of piling, brush and heavy rip-rap answers the purpose. The jetty created is, practically, an anchored sandbar. It will be well for engineers to take this lesson to heart. The forces of Nature have been mastered economically by pitting them against each other.

The great principle so far evolved in harbor physics is that a channel across the ocean bar must be created and maintained, wholly or in part, by increasing the force of passing tidal currents. Time and again this has been attempted with a single jetty or with drowned jetties, but failure has generally resulted. Where single jetties have been successful, it will probably be found that existent conditions supplied the other wall to the created channel. As for the drowned jetty—a device which throws away a large portion of the ebb forces, while it scatters those of the flood—it has taught a great lesson. It seems best now to utilize all of the known forces which favor a jetty project, because counter forces may be developed which cannot be foreseen. A margin on the safe side is always more or less comforting to man battling with powers seen and unseen.

And here it may be observed that more or less mathematical analysis is sometimes wasted on the discussion of harbor problems. But the application of rigid logic to ill-defined and unknown elements cannot be satisfactory in results. The water volumes depend upon meteorological conditions and these depend upon many elements that we are not now able to predict or measure.

At the mouth of the Columbia River the single jetty is reinforced with conditions which partly supply another. Had those conditions been utilized by throwing in a cheap structure from Chinook Point to Sand Island, the enormous tidal currents might reasonably have been expected to cut through the bar with considerably less of south jetty.

A considerable volume of the ebb tide used to pass through Baker's Bay around to the north of Sand Island where it became concentrated between the latter and Peacock Spit and was thrown at right angles, nearly, into the main volume relied upon to do the scouring. This effect has been minimized to a great extent by an adventitious circumstance. The large schools of salmon which annually run into the Columbia River from the ocean use Baker's Bay as a resting place where they get somewhat acclimated to the fresh water. Taking advantage of this circumstance the fishermen have literally filled the bay with pound nets stretching from the shores and shoals into the deep water at about right angles to the tidal currents. These, acting as permeable groins, have largely filled up Baker's Bay and sufficiently closed the gap in the natural north jetty for the purposes of the south jetty as constructed.

General D. S. Stanley.—I have been in Texas for the last dozen years and have seen the growth of the jetty at Galveston, which is the most extensive jetty work that has been undertaken in the United States. The appropriation is over $6 000 000, and the contract given out under that appropriation is one of the largest contracts ever given out in the United States.

The harbor at Galveston is about 2 miles wide; the inner bar had ordinarily, say 15 or 20 years ago, about 17 ft. of water, and the outer bar about 11 ft., at high tide. The kinds of vessels that could enter were very restricted. The appropriations were small 10 years ago, and the first attempts at improvements were what might be called experimental. They had $35 000 or $40 000 a year, and tried caissons and various similar experiments to control the tides, but without any success at all; but afterwards the appropriations were raised to as high as $300 000 and $350 000 a year. The engineers tried to put down mattresses weighted down with stone; the mattresses were 150 to 200 ft. long, 70 ft. wide, and there was an immense amount of money spent on them. The brush had to be brought a very long distance, and the construction of the platforms to build them on was very expensive, and that whole work was utterly useless; it all went to pieces from the ravages of the teredo, or sea-worm, as soon as submerged. The engineer in charge was very sanguine that, as soon as these mattresses were put down, the wave action and the tides would cover the brush with sand, and, as the teredo cannot thrive where he is covered with sand, that the mattresses would prove permanent. Those mattresses disappeared, and when the engineer who next took charge finally settled upon a plan of using nothing but rock, he told me that he did not find anything of the mattresses left. I have seen the butt end of the brush, which composed the mattresses, as thick as the wrist, that could be crushed in the hand. The engineer in charge came to the conclusion, finally, that they could build a jetty there on the same plan that big buildings were built in the City of Galveston. In putting up a large building there, they simply sweep the ground off, and commence with the foundation, and run up to six or seven stories high. These buildings are always secure so long as the water does not touch the foundation; but if a tin spout is out of order, and the water is allowed to run down to the foundation a single night, the house may tumble down before morning.

This is a longer jetty than the jetty at the mouth of the Columbia. It was constructed by running a trestle of timber 1 ft. square in cross-section, driven down strongly by pile-drivers put fairly in advance of the work. They found that it did not do to put this trestle much in advance of the stone-work. The core of the jetty is built of sandstone. It was very difficult to get a sandstone in Texas that had the necessary weight. After the trestle is built, the stone is tumbled off the cars, and let take its shape as it tumbles down. Then this is encased with a strong coat of granite, brought from nearly 300 miles away. The blocks are increased in weight as you go towards the end of the jetty; at the end of the jetty there are no granite blocks which are less than 10 tons each. The work is now nearly completed; but there is this difficulty with the extreme end of it, that the currents around the extreme point are unsettling the end of the jetty. The contractors complain that this puts them to excessive expense. But the result in deepening the harbor is hardly satisfactory; thus far, the best they can show is about 16 ft. That is, of course, a great advance from 11 ft. The purpose in bringing this long jetty out from Galveston Island is that they propose to build a jetty 2½ miles from the main land at Bolivar Point. This would be convergent to the long jetty from the island. The points of those jetties are about 3 000 ft. apart, and that was the great subject of dispute between the engineers and Mr. Eads. Mr. Eads proposed the points of the jetties should be 600 ft. apart; the engineers all wanted a low jetty. A very able engineer said to me that it would be a crime to put in a high jetty, because the current would strike right across the City of Galveston and take that out to sea.

I would like to ask Captain Black if he has had similar trouble with the teredo.

William M. Black, Am. Soc. C. E.—I have, often. When we first started in at the St. John's we used the mattresses, and it was found that none but the lower mattress got coated with sand; all the rest were eaten out. The teredo punctured all of the lower mattresses.

Charles Macdonald, M. Am. Soc. C. E.—Is not the principal difficulty at Galveston the fact that there is an enormous body of water inland, which gives a different condition of currents?

General Stanley.—Galveston Bay is a large body of water; there is a surface of at least 300 sq. miles in Galveston Bay and the highest

tide is only 4 ft., so that the current from that tide must necessarily be light.

Mr. Macdonald.—During a visit at Galveston some years ago, I had, from the United States Engineer officer in charge of the improvements of that harbor, a very instructive description of the chief difficulty with which they had to contend in maintaining a deep channel over the bar.

It appears that during continued heavy storms from the south, the waters of the Gulf are forced through the inlet, causing the waters of the inner bay to rise rapidly to the northward.

It will be remembered that upon one such occasion, when all telegraphic communication with Galveston had been cut off, it was reported from the northern end of the bay (about 80 miles distant from Galveston) that the city itself must be entirely under water, as the bay at the upper point had risen 20 ft., while the general level of the City of Galveston is but a few feet above the Gulf.

This report proved to be untrue ; but, upon the subsidence of the storm, the water which had been forced into the bay came down with great rapidity, and carried away a considerable portion of the jetty work which had already been done at the inlet. The necessity of providing for such contingencies makes the problem of harbor improvement at Galveston an exceedingly difficult one.

General Stanley.—Yes, the danger there is from the southwest storms that may push the water up into the Galveston Bay ; then if you have a sudden "norther," it goes down in a great flood. In 1875, just above the quarantine station, it cut a channel 11 ft. deep. That was open for five or six years. That is the danger of putting in a high jetty which would stop this flow ; but with a low jetty, the theory is that the water will pass over the top without any harm to the city. They have had several severe storms since the jetty was put in, and they feel more confident now.

A. J. Frith, M. Am. Soc. C. E.—Captain Black has referred to the question of piles cutting out very severely at the foundations. This experience is similar to that on the western rivers, where great difficulty was met in holding pile dikes. No matter how strong a pile dike is made, or how heavy the mattress is at the bottom, if there is a heavy current passing down the chute, there will be eminent danger of its being undermined. Of course, we all understand that the water

flows down from above, due to the difference of elevation at the two ends of the chute, and that its energy is very great. If, then, we build a pile dike, or an obstruction of that character, it necessarily contracts the area at that point, the velocity is increased, and the water will force itself beneath almost any mattress which is put down. I think from the experience on the rivers there it was found that much better results were obtained when the dikes were considerably down the chute. There was no attempt made to raise an obstruction crosswise with such dikes, but that effect was obtained by the collection of drift in front of the dike, the attempt being not to stop the flow of water directly, but to interpose a frictional or skin resistance to the flow of the water, due to the heavy mass on the surface or floating raft. In a number of instances in that locality the currents have been entirely turned aside. In one case simply by the action of the fleet of boats in resisting the flow of the water, a heavy deposit in the whole chute was rapidly attained. Where the piles were held, the difficulty was with the material of the piling; the life of this material was very short, so that after a few years the obstruction in a great many cases gave way. If that had been obviated, and those drift rafts had been held, in all cases the current could have been entirely changed and driven out. I have seen one raft there that was certainly as much as 60 acres in extent, and it was so interlaced with all the débris that came down the river that it was truly a solid mass; and as the river fell this became imbedded in the mud, and when the river rose again only a part of it came up. Our experience at that time was that this was much the cheapest and safest method of stopping the flow of a chute; but where it was attempted to stop it by the actual obstruction of piling, that the entire head of water in the chute was concentrated at the dike itself, and it might scour down to almost any depth.

Captain Thos. W. Symons.—Mr. Wisner very properly notices the discrepancy in the actual cost per foot of the work at Yaquina Bay and the cost as determined by the various items entering into the structure at the stated cost price. In explanation of this discrepancy, it should be remembered that the cost of items named is only that which prevailed during the past year, with everything in the way of adequate plant on hand, all auxiliary work done, and appropriations sufficient to carry on the work continuously. The stated cost of items takes no account of the cost of plant, the expense of opening quarries, etc. It is the

bare cost of labor and materials expended. As stated in the article, the actual "cost of the various items entering into the work has varied greatly."

This is one point in explanation. The most important cause of the discrepancy is, however, due to inadequate appropriations, scattered over a long period of time. With the small early appropriations the officers in charge tried to and did produce results of immediate value, with structures which afterward turned out to be lacking in strength and suitability to the location. From this cause, the work done with the early appropriations plays but a small part in the whole structure as now about completed.

The lack of money to carry on the work continuously has been the cause of great expense in laying up the plant, caring for it and the works during the interim, and putting things in shape and starting up again. The length of time, 12 years, during which work has been carried on, has directly resulted in great expense, owing to the deterioration and destruction of the auxiliary structures necessary in connection with the jetties. A large part of the south jetty tramway has been built over twice, and a very considerable portion three times. For the south jetty four wharves have been built, the accumulation of sand about the first three having rendered this necessary. The fourth is 4 400 ft. from the first one.

A large amount of work had to be done from time to time to protect the south spit from erosion and to protect the tramway approaches from injury by drift. Mr. Wisner overlooks entirely the mattress work of the south jetty, which cost a large sum.

Added to the above is the cost and repair of plant, cost of lands, quarries, etc., cost of frequent surveys, former high cost of labor, and difficulty of obtaining it; almost total lack of skilled labor and experience with such work and the general sum total cause of the discrepancy are stated.

This work has been the school of experience in which valuable lessons have been learned, which lessons are being made use of at various points along the coast.

The cost of the Columbia River work has been far below the original estimate, not only because some of the more expensive concrete work has been omitted, but because of the excellent method adopted for carrying on the work, the suitability and efficiency of the plant

designed and built for carrying it on, and the high administrative ability displayed in conducting it. It is to the engineering skill and ability of those having the work in charge that the reduction of cost has been reduced, and this is a truth which I feel at liberty to emphasize, never having personally had anything to do with it. The reduction in cost of the rock has not been anywhere as great, however, as mentioned by Mr. Wisner, from $5 per cubic yard in the estimate to about one-fifth as much.

The estimated cost of $5 per cubic yard included everything.

Counting the cost of everything and only considering the rock in the jetty, the latter has cost about $3.50 per cubic yard, instead of about $1, as inferred by Mr. Wisner.

I did not specify items of cost more explicitly, as mentioned by Mr. Wisner, because it is my experience that cost varies so much, due to local circumstances, that such data is of comparatively little use unless all the local circumstances are fully known.

By the further extension of the Yaquina Bay jetties 2 000 ft., as mentioned by Mr. Wisner, the bar would be driven out to about the line of the outlying reef, and all the advantages of this reef as a breakwater would be lost. It is doubtful if such an extension would be of the slightest benefit. It is quite possible that it would be decidedly detrimental.

I am unable to appreciate the advantages claimed for long parallel jetties by Mr. Wisner of producing greater scour on the bar due to the momentum acquired by the waters in the direction of the channel axis. To my mind, the contracted parallel portions should be as short as possible consistent with giving the desired direction to the flowing waters, and these waters should arrive at the contracted portion of the channel with the greatest available ease. The case is somewhat analagous to that of a fire stream through hose and nozzle; the desired effect would not be increased by increasing the length of the nozzle. There are some decided advantages of parallel jetties, but I cannot believe in that pointed out by Mr. Wisner.

Since writing my paper the groins mentioned on page 180 have been added to the south jetty at Yaquina Bay. Three of these are 100 ft. long, and two are 60 ft. long.

In case it ever becomes necessary or desirable to reinforce the jetties at the Columbia River or elsewhere with large concrete blocks,

there is very little doubt that the method of building them *in situ* mentioned by Mr. Wisner will be considered.

In regard to Mr. Wisner's closing paragraph, it may be stated that it is the practice to follow up the pilework as rapidly as possible with the mattress work. As the tramway cannot be braced until the center mattresses are down, it is desirable to get them down as soon as possible, not only to prevent scour, but to enable the strengthening and stiffening braces to go in.

Major Jones' graphic comments emphasize the fact that is ever before us, that in designing and carrying out works on this coast it is essential to work in harmony with Nature.

In regard to the destruction of mattresses by the teredo, I will state that it is doubtful if any mattresses have ever been injured on this coast by this pest. At the Columbia the fresh water prevents it working; at other points, where it is active, the movement of the sand is so rapid that a mattress is no sooner down and covered with rock than it is buried in sand. The only damage done by the teredo is to our piling.

Wm. M. Black, M. Am. Soc. C. E.—Regarding the efficiency of the converging jetties in the harbors described, I would invite attention to the map of Yaquina Bay, and particularly to the accompanying profile. It will be noted that the ends of the jetties are located on the crest of the bar as it existed before improvement, and that the jetties converge to this point, the width between them there being the minimum width proposed for the entrance; it will be noted also that the jetties are nearly straight, and that, starting from safe locations on the shores of the entrance, they follow nearly the shortest lines to the points where the work was required, thus securing the desired result with the least length of jetty, and, therefore, with the least probable cost, so far as the cost depends on location.

In general, in Government jetty work on the sea coast, for drift and wave bars, lying at some distance outside the coast line, the jetties are started from safe points on the two sides of the entrance and converge to a point at a distance from the shore fixed by the length through which the uncontrolled and unaided currents from the entrance can maintain the desired depth. When extended beyond this point they are nearly or quite parallel. The contraction is made only where necessary. If the jetties follow proper lines, the formation of an interior bar need not be feared.

Economy in construction methods is only possible where the necessary funds are continuously available. The methods followed on the Atlantic Coast, where the mattresses and stone are towed to the site of the work, require but a comparatively inexpensive plant, which can be left idle through long periods without much loss. Where jetties are built from overhead tramways exposed to storm, wave action and the teredo, the first cost is very great and the deterioration of plant (piles weakened by teredo, rails corroded, tramway injured by storm action) so rapid as to make this method inadvisable unless work can be prosecuted vigorously and continuously.

A marked difference between the Pacific and Atlantic coast jetties is found in the width of mattress foundation used. On the Pacific coast a width of about 40 ft. has been found sufficient. On the Atlantic coast the width is much greater. At the mouth of the St. John's River the width finally adopted is 120 ft. This great width was found to be necessary to prevent undermining. The edges protruding beyond the base of the superstructure act as an apron, broadest in shoal water, to receive the downpour of water from waves breaking across the jetty. The width of foundation must be governed by the degree of fineness of the sand on which the jetties rest and by other local conditions.

The attempt to take advantage of existing sandbanks for the purpose of locating jetties in shoal water, and thus economizing jetty height, is almost always, to a great extent, futile. The new conditions introduced by the solid body will produce scour, if there be any current existing, or induced by the jetty. Scour in a soft bottom in advance of the jetty is almost invariable, and, if the jetty extension be delayed, a deep hole will generally form at the end of the completed work.

The efficiency of piles for producing scour, where they are not so closely placed as to stop the current entirely, is well known, and in the Mississippi River temporary channels have been formed across bars by the simple expedient of driving a row of piles, spaced 6 to 10 ft. apart, along the line of the proposed cut. Where a pile tramway is built in advance of the jetty, the scour becomes very rapid, and the protection of the bottom by mattress work should follow close after the tramway construction.

This scour in advance of the jetty is not altogether an evil, for

should a deep channel form close beside a jetty built in very shoal water, unless the mattress foundation protection be very broad, the destruction of the jetty may follow from side undermining.

The relative value of high and low jetties is yet a mootable question. A low jetty guides a current moving in a channel parallel to its axis and keeps it in its proper course. If there is a continuous escape of water across the jetty crest, the head available for work further on is continuously reduced, and the current may become too weak for the duty required of it, *i. e.*, for keeping the channel clear to deep water beyond the end of the jetty. A submerged jetty is little affected by wave action and can be built of cheap material. The higher the jetty, the more material it contains and the more it must be prepared to resist the force of the waves, and therefore the stronger must be its structure. Where the jetty is so high as to prevent waves from crossing it, the action below water becomes more violent. In submerged jetties on the Atlantic coast, stones of 200 lbs. weight will be stable at and below the level of low water, while with high structures, authorities seem to agree that more or less disturbance is to be expected to a depth of 25 ft. below that level.

On our South Atlantic coast the growth of shell fish materially assists in increasing the strength of the jetties. At the mouth of St. John's River I have seen stone which had been in the water not more than six months completely covered with small oysters and barnacles.

Other unsettled questions in our coast jetty work are whether two jetties are necessary, or, if one jetty, on which side of the entrance it should be placed. On the South Atlantic coast the bars are formed by coast drift having a resultant movement from north to south. This movement causes the channels from the openings in the coast line to shift to the south, until, becoming too long, the difference of head between the inside water and the ocean at low tide develops sufficient power to break a new and shorter path to the sea, when the slow shift to the south again begins. The changes in channel depth take place on the bar. Close in shore there is usually a permanent deep pocket extending from the entrance in a southerly direction, which forms a root, as it were, from which the shifting channels spring.

My understanding of the action of the jetties on this coast is as follows. A north jetty protects the temporary channel straight seaward from the north end of this pocket, and it becomes more per-

manent, but under winds from the north and the slight southerly tendency of ebb tides there is an escape of water to the south, and the north channel does not acquire great depth. If by storm action, sand is swept across the north jetty, this north channel may be shifted or, temporarily at least, obliterated.

A single jetty on the south side, straight or concave to the current, prevents the escape of water to the south and guides the current in the desired direction. The sand movement from the north holds the mass of the water flowing to and from the entrance close against the jetty and maintains a channel there, but the sandbanks on the north are not sufficiently high or sufficiently stable. There is always an escape of water across them through minor channels. In the unimproved harbors there is generally a flood channel close along shore on the north side of the entrance through which a portion of the ebbs escapes. Under the influence of a long-continued southeast gale so much water may be driven through these secondary channels as to reduce the flow through the south, or jetty, channel below the volume necessary for maintaining this channel at full depth, resulting in the temporary closing of the port.

It is my belief that under the conditions prevailing on the South Atlantic coast, two jetties are required to produce and maintain a permanent deep entrance to the ports, it being understood that they shall be located and built with due regard to the essential condition that the tidal prism of the harbor must not be reduced. I am not familiar enough with the physical conditions of the Pacific coast harbors to judge of their requirements. Apparently these conditions differ only in degree from those of the Atlantic coast. At the mouth of the Columbia River the volume of flow is very great and the rocky headland on the north acts as a short jetty, so that it is possible that, while the ebb flow may be scattered through several channels, there will remain a volume of flow through the main channel sufficient to maintain the required depth without further jetty construction.